AD-A134 708 CONSTRUCTION FOUNDATION REPORT MISSOURI RIVER FORT PECK LAKE MONTANA VOLUME 1 TEXT AND PHOTOS(U) ARMY ENGINEER DISTRICT OMAHA NE JAN 83 1/2

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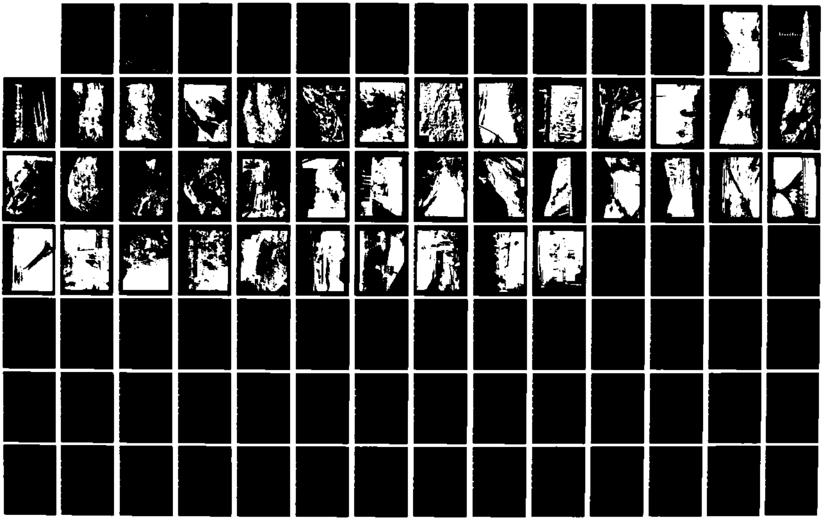

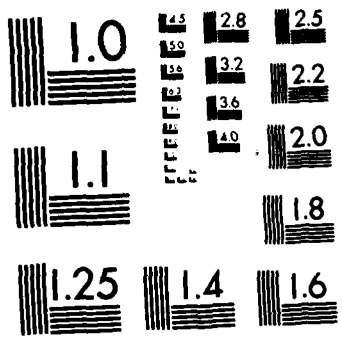
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CONSTRUCTION FOUNDATION REPORT

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AD-A134708

MISSOURI RIVER
FORT PECK LAKE, MONTANA

VOLUME I
TEXT AND PHOTOS

JANUARY 1983

US Army Corps of Engineers
Omaha District

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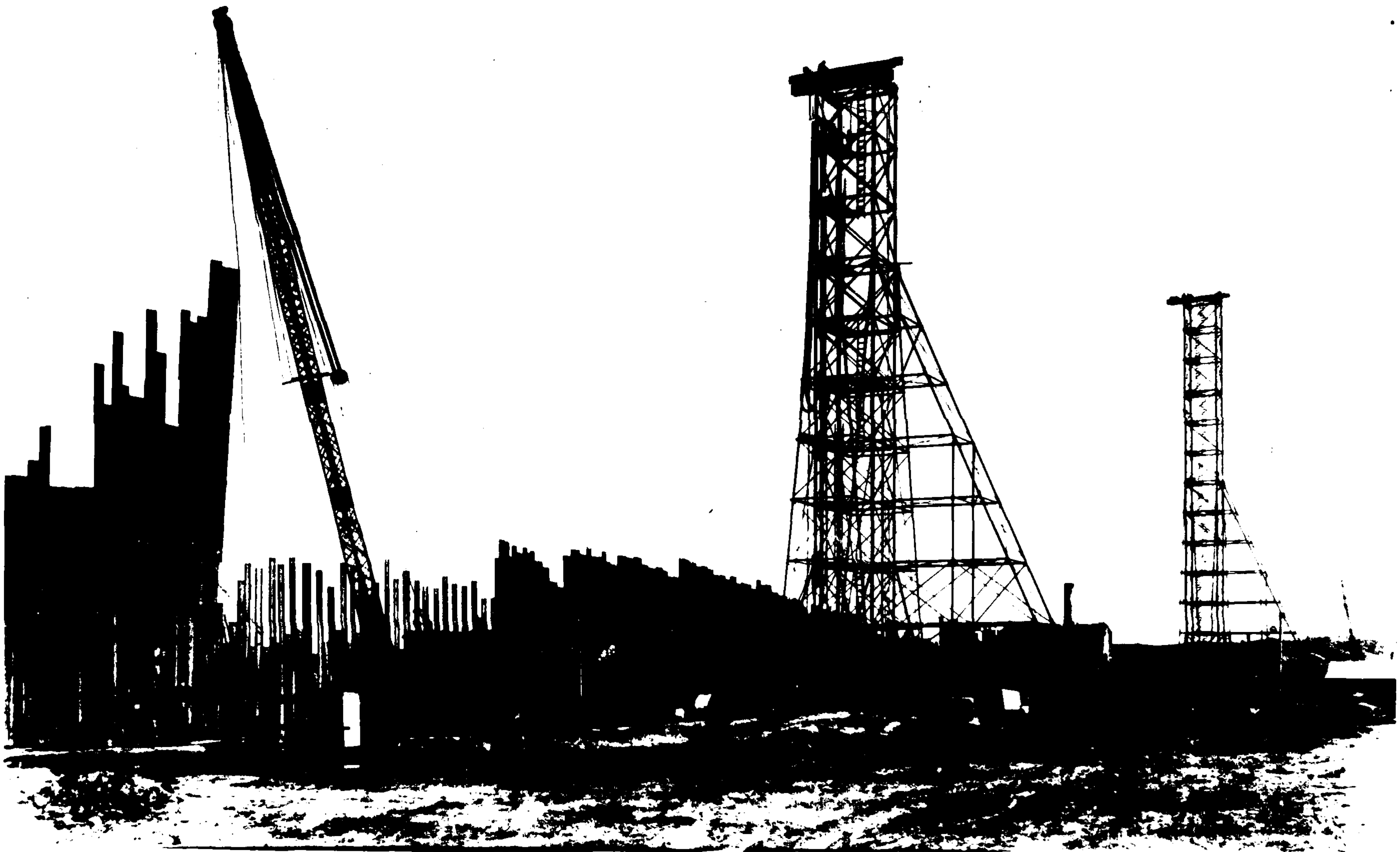

Photo No. 1A

FORT PECK DISTRICT CORPS OF ENGINEERS DEPARTMENT OF THE ARMY FORT PECK, MONTANA

FORT PECK DAM AND RESERVOIR PROJECT PHOTOGRAPH NO. 34/1269 20 October 1934

View looking east showing steel sheet pile cut-off wall being installed. Locomotive crane shown at left, 150 foot gantry crane in center, and 196 foot gantry crane at right.

PLATE NO. 9

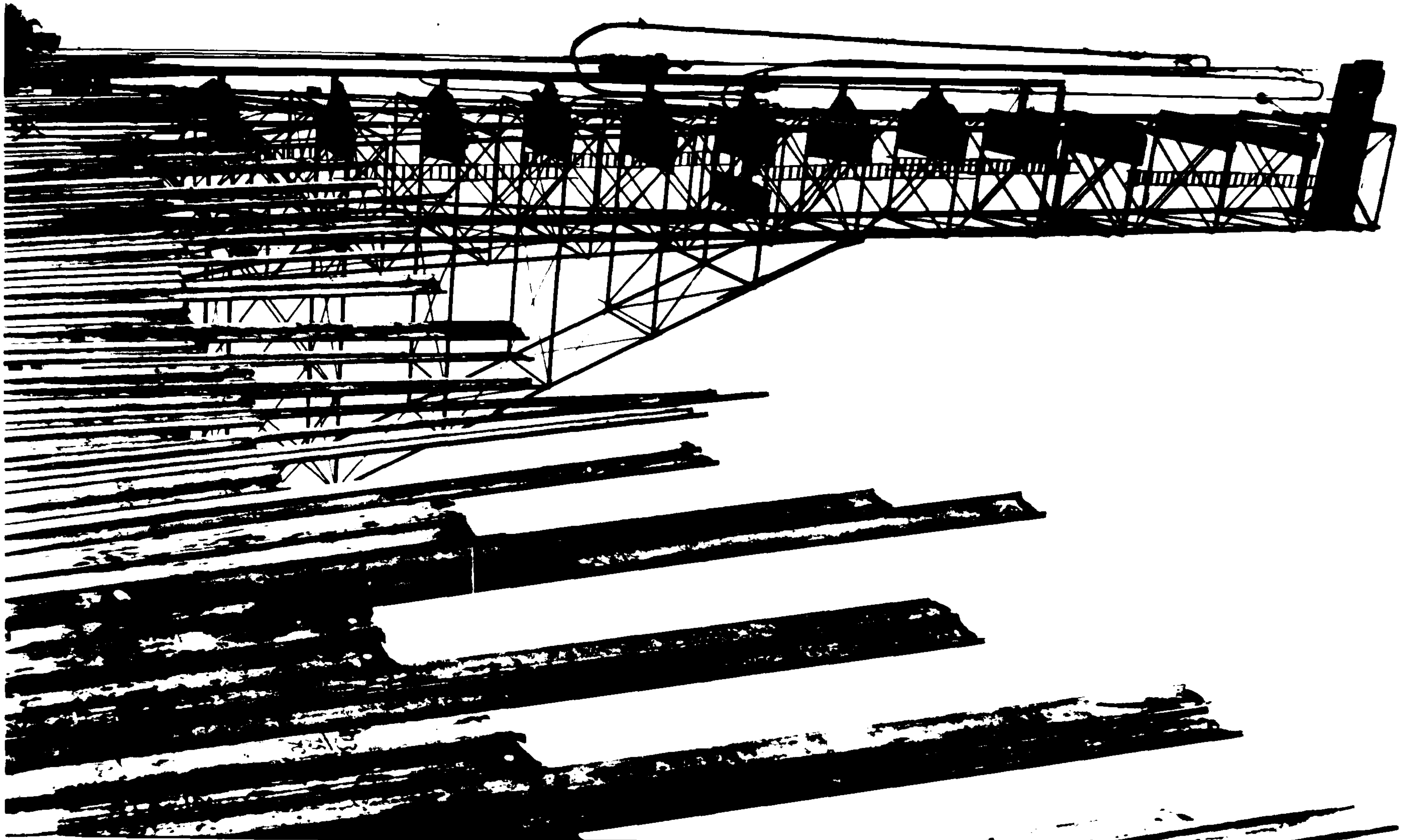

Photo No. 1B

FORT PECK DISTRICT CORPS OF ENGINEERS DEPARTMENT OF THE ARMY FORT PECK, MONTANA

FORT PECK DAM AND RESERVOIR PROJECT PHOTOGRAPH NO. 35/2164 17 September 1935

Close-up view of 196 foot gantry crane driving steel sheet piling cut-off wall.

PLATE NO. 10

Photo No. 1C

FORT PECK DISTRICT CORPS OF ENGINEERS DEPARTMENT OF THE ARMY FORT PECK, MONTANA

FORT PECK DAM AND RESERVOIR PROJECT PHOTOGRAPH NO. 36/2479 18 April 1936

View of hydraulic spade jet used in installation of steel sheet pile cut-off wall.

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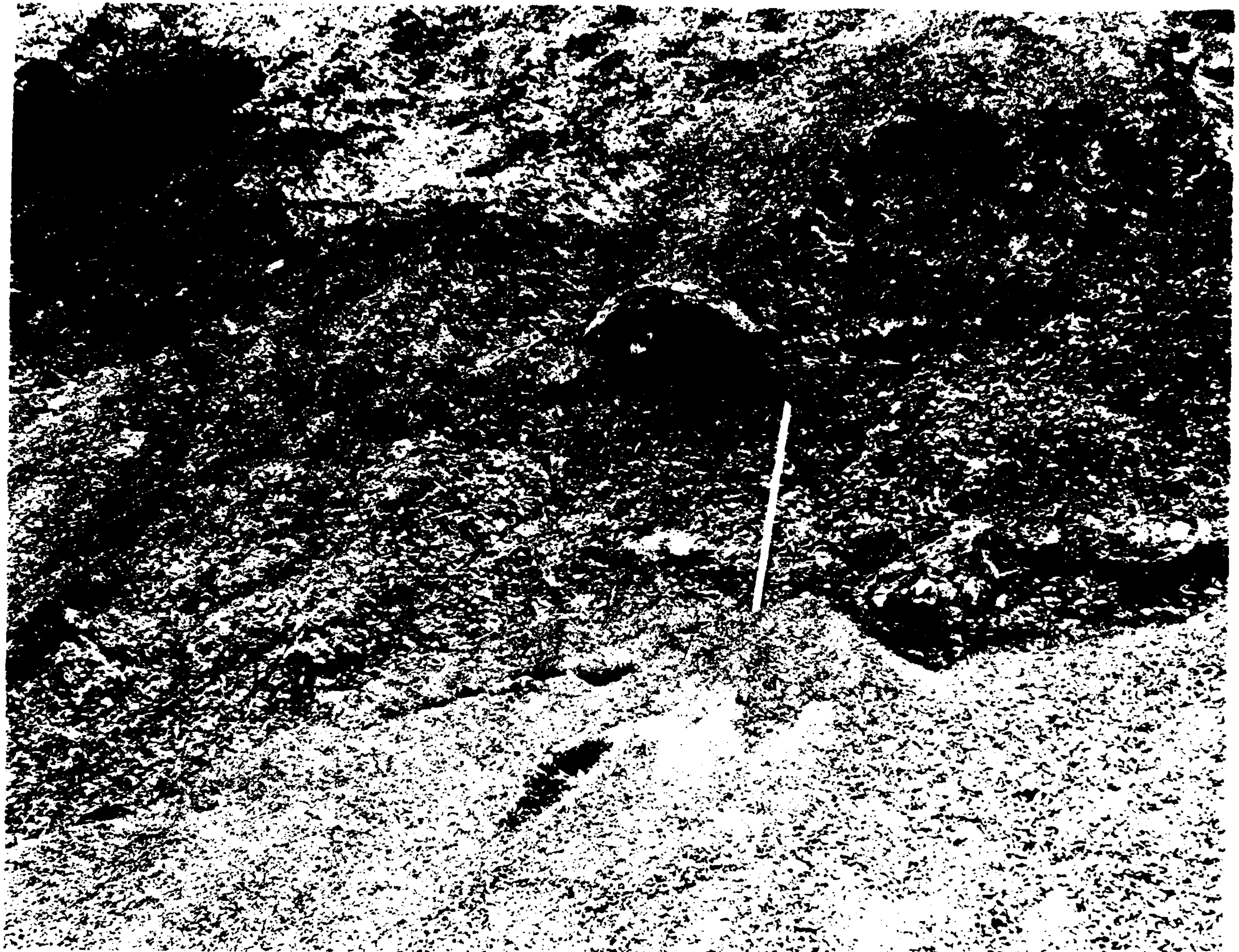

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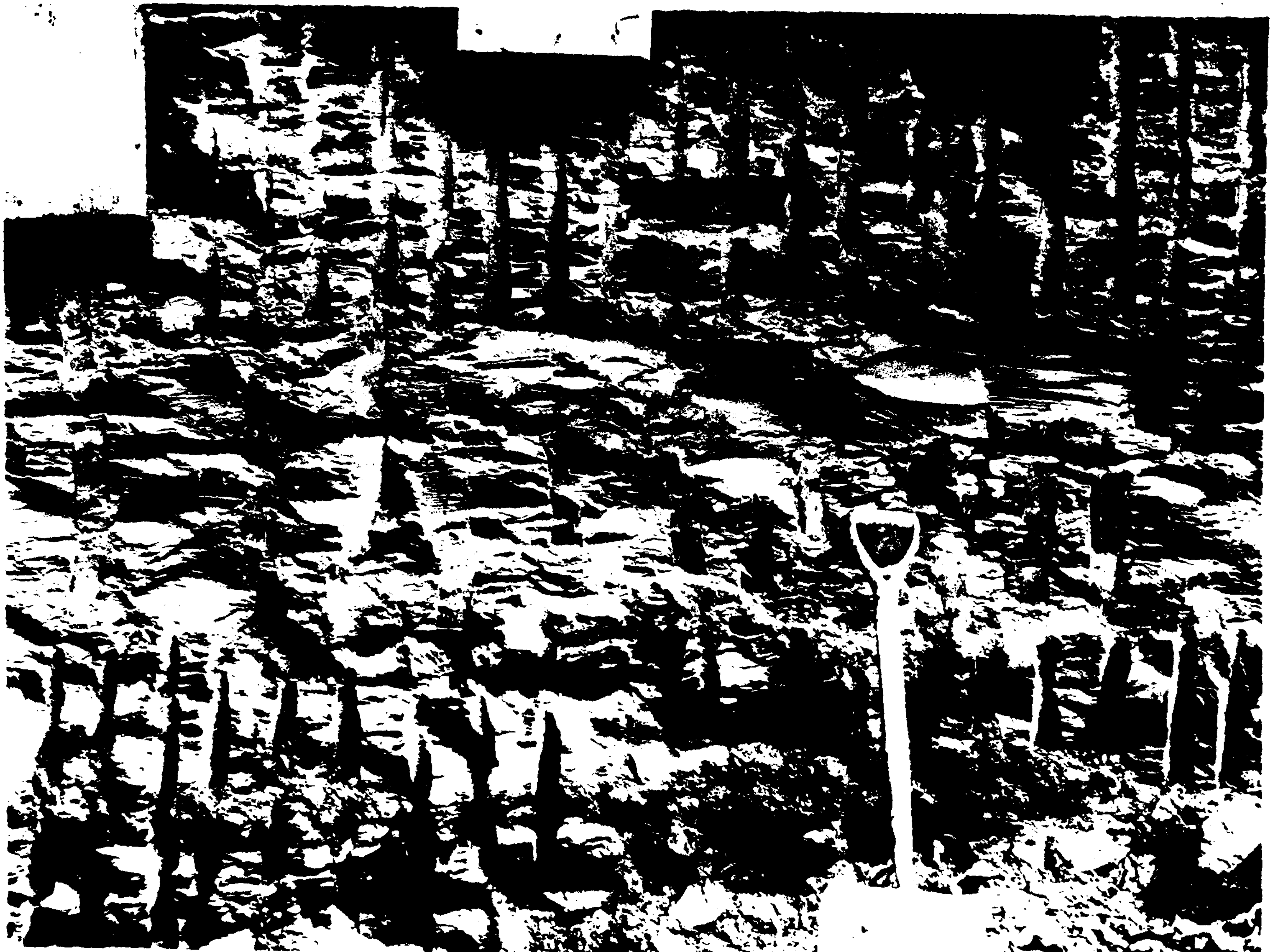

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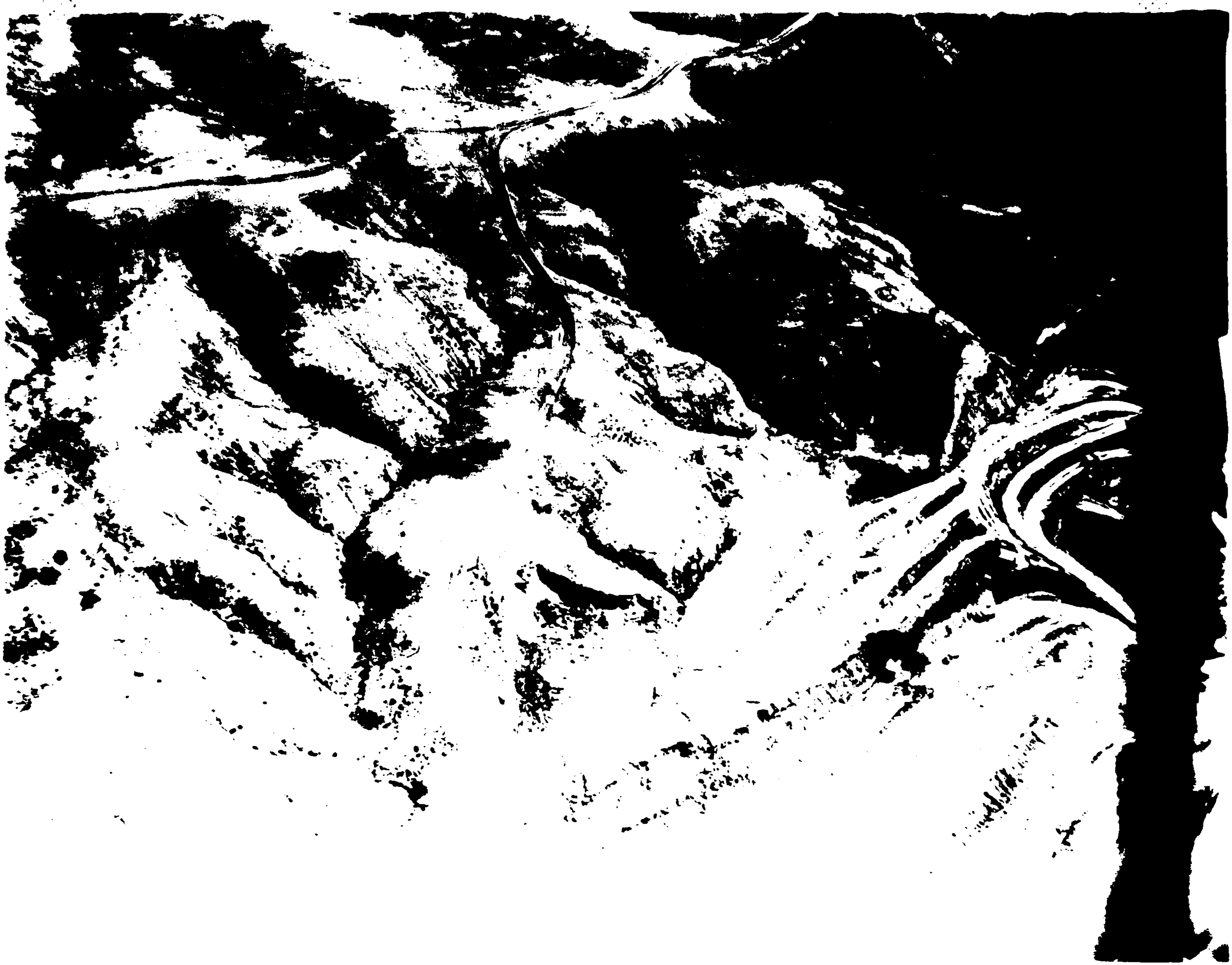

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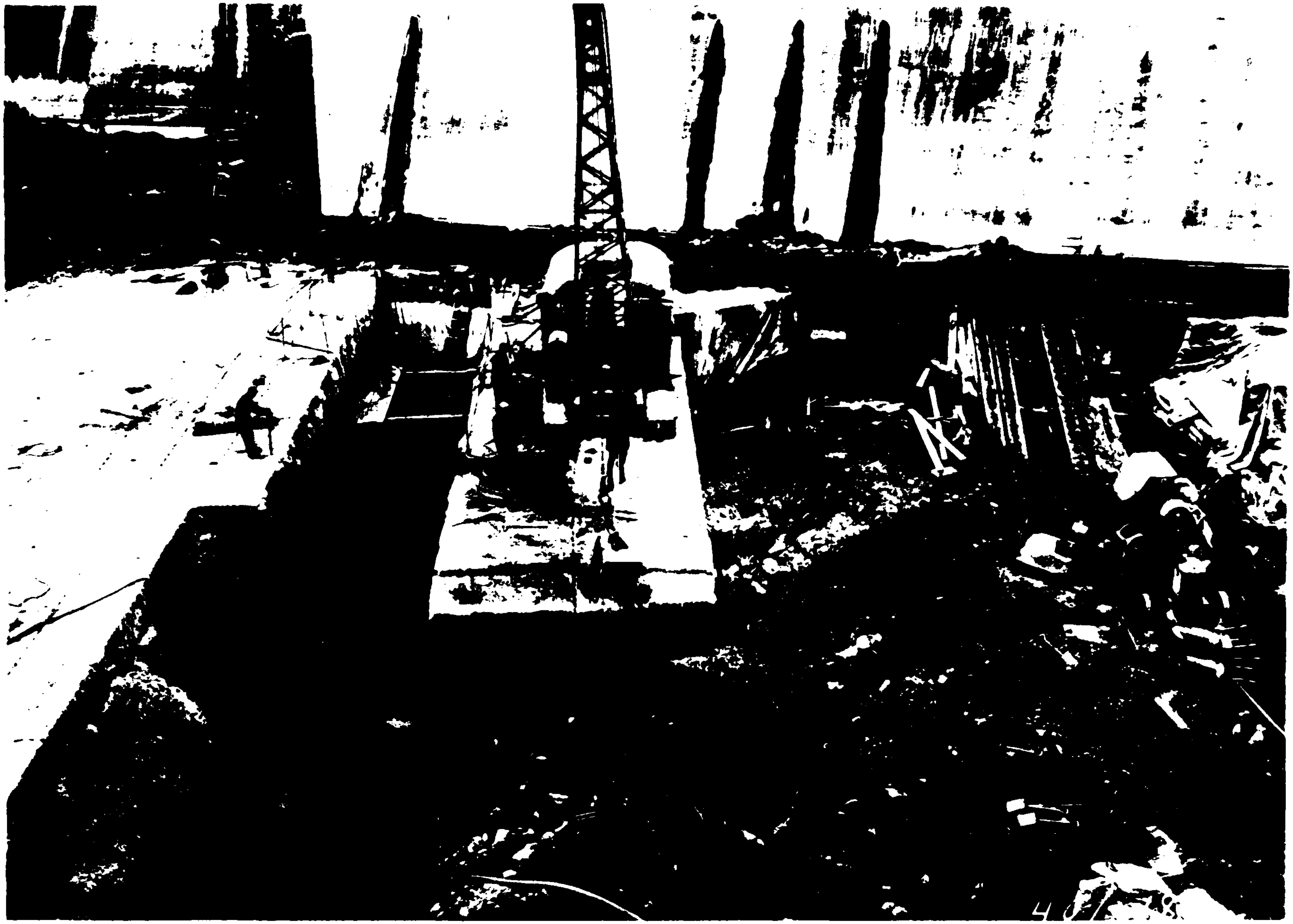

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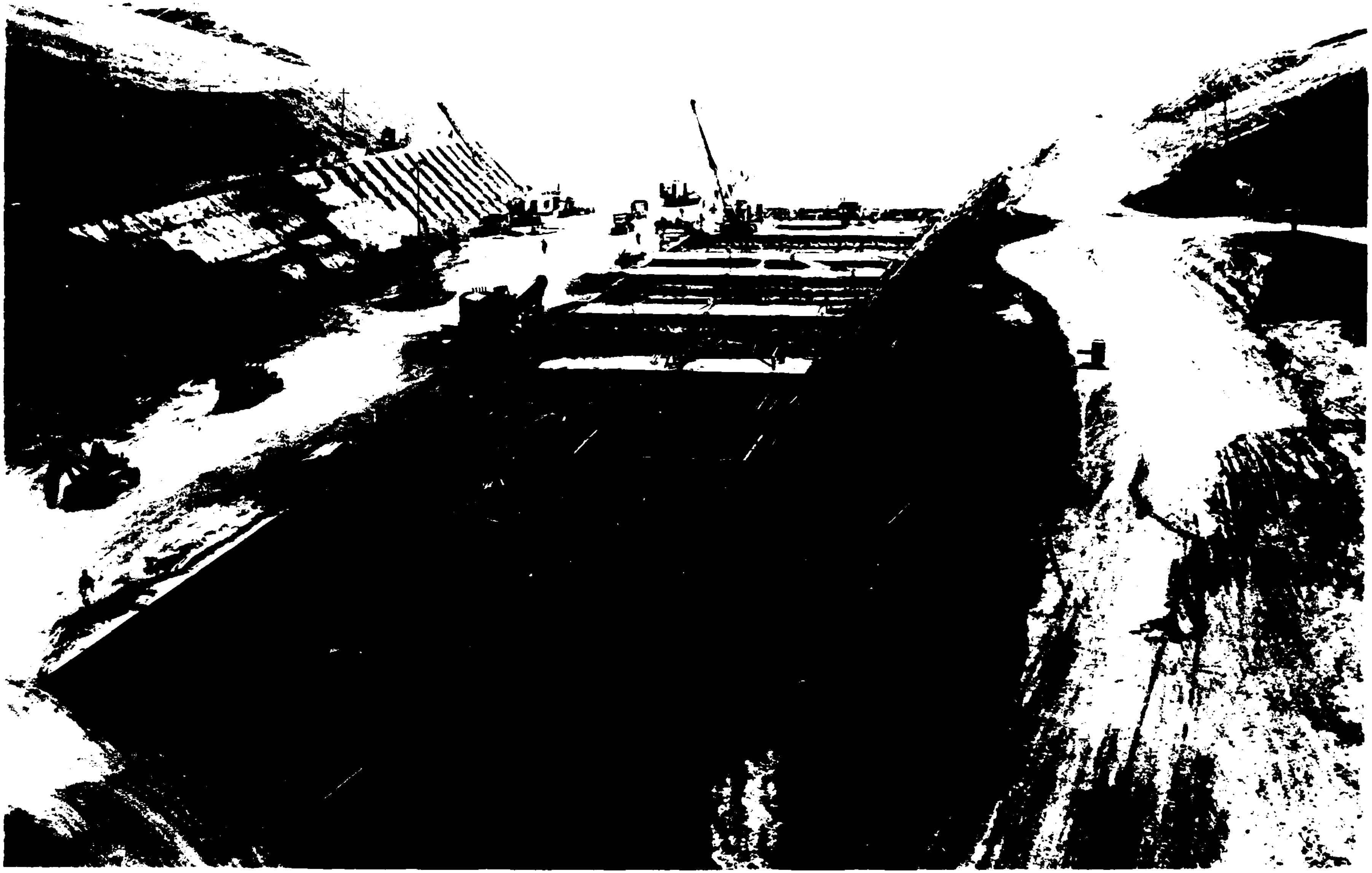

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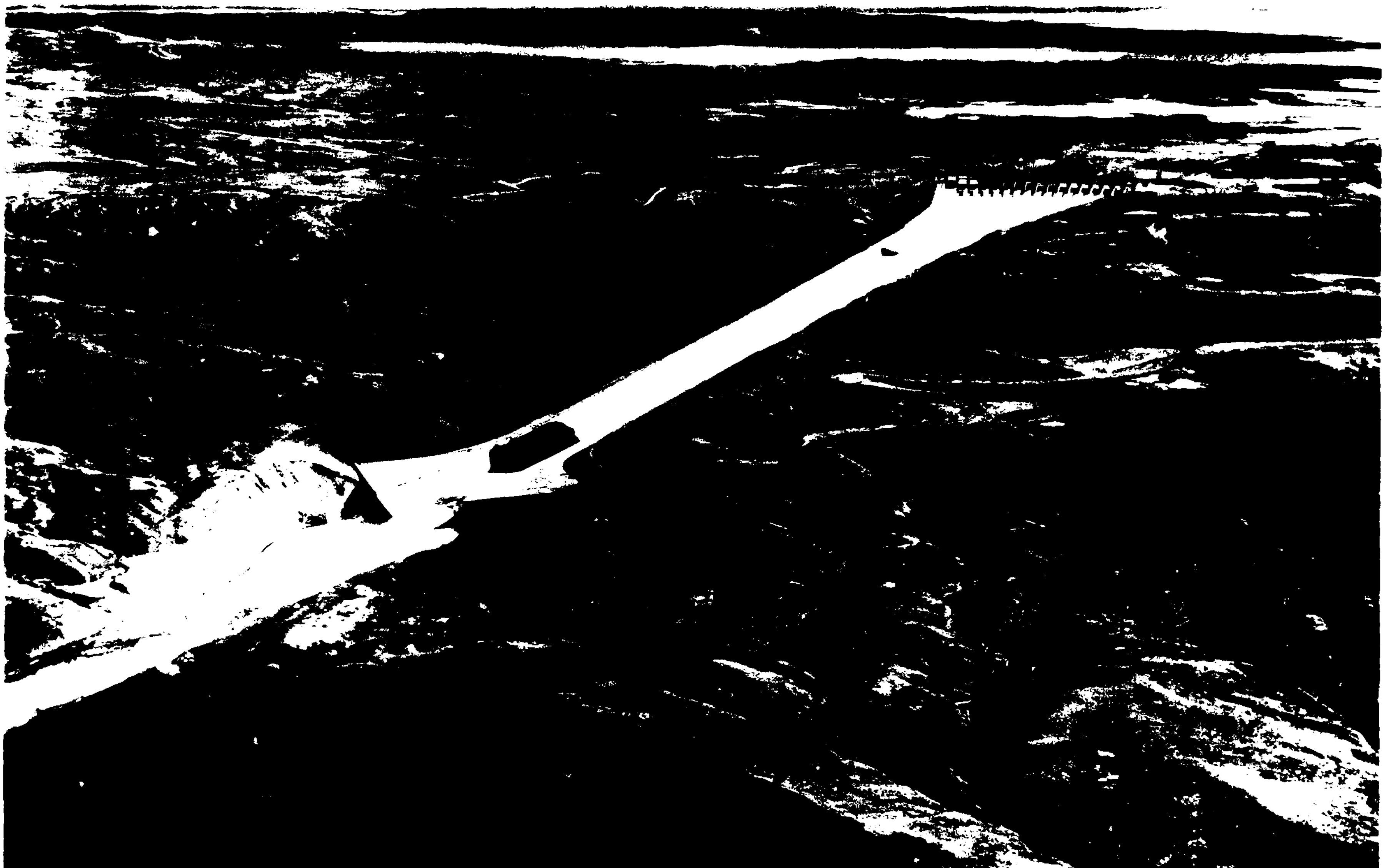

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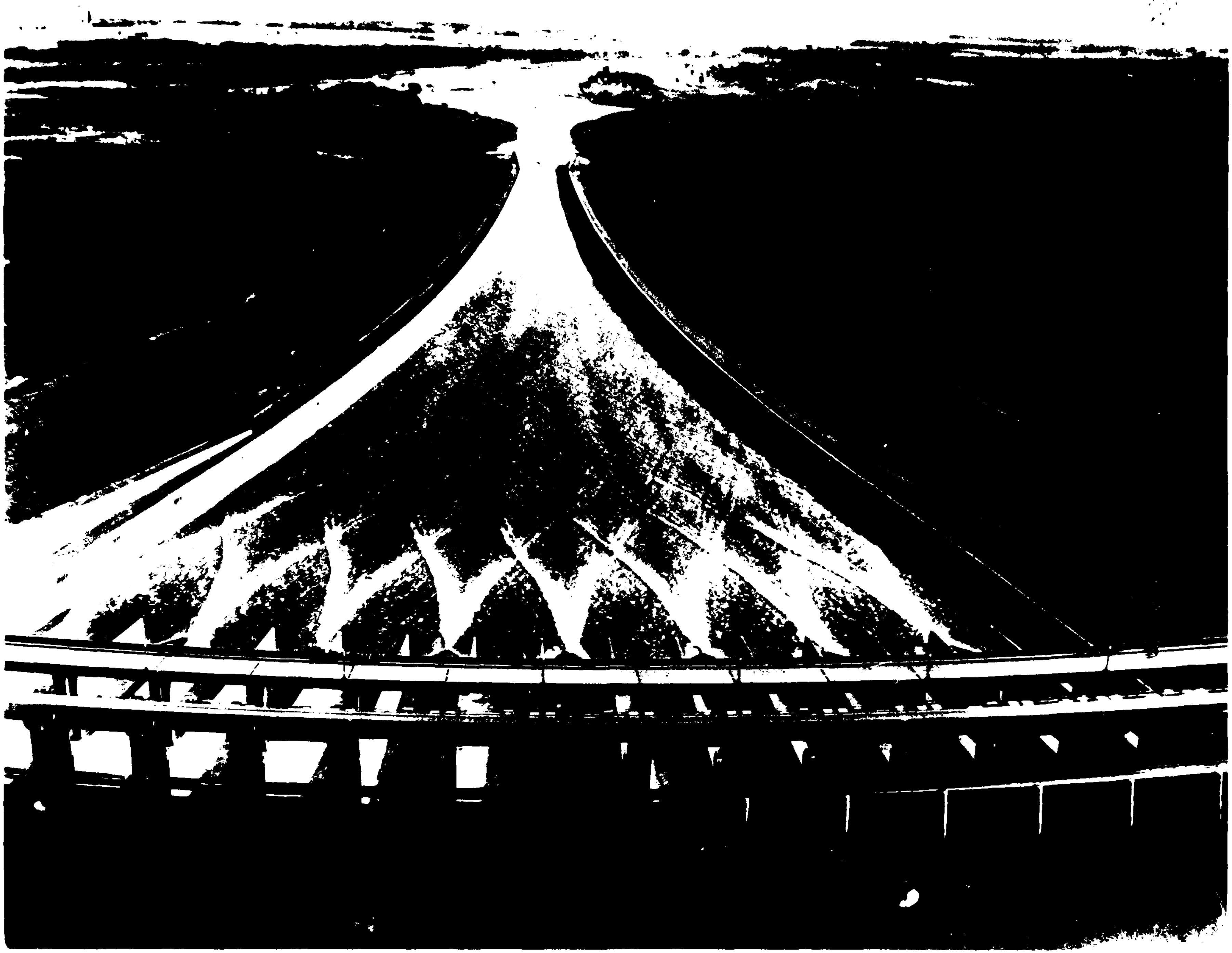

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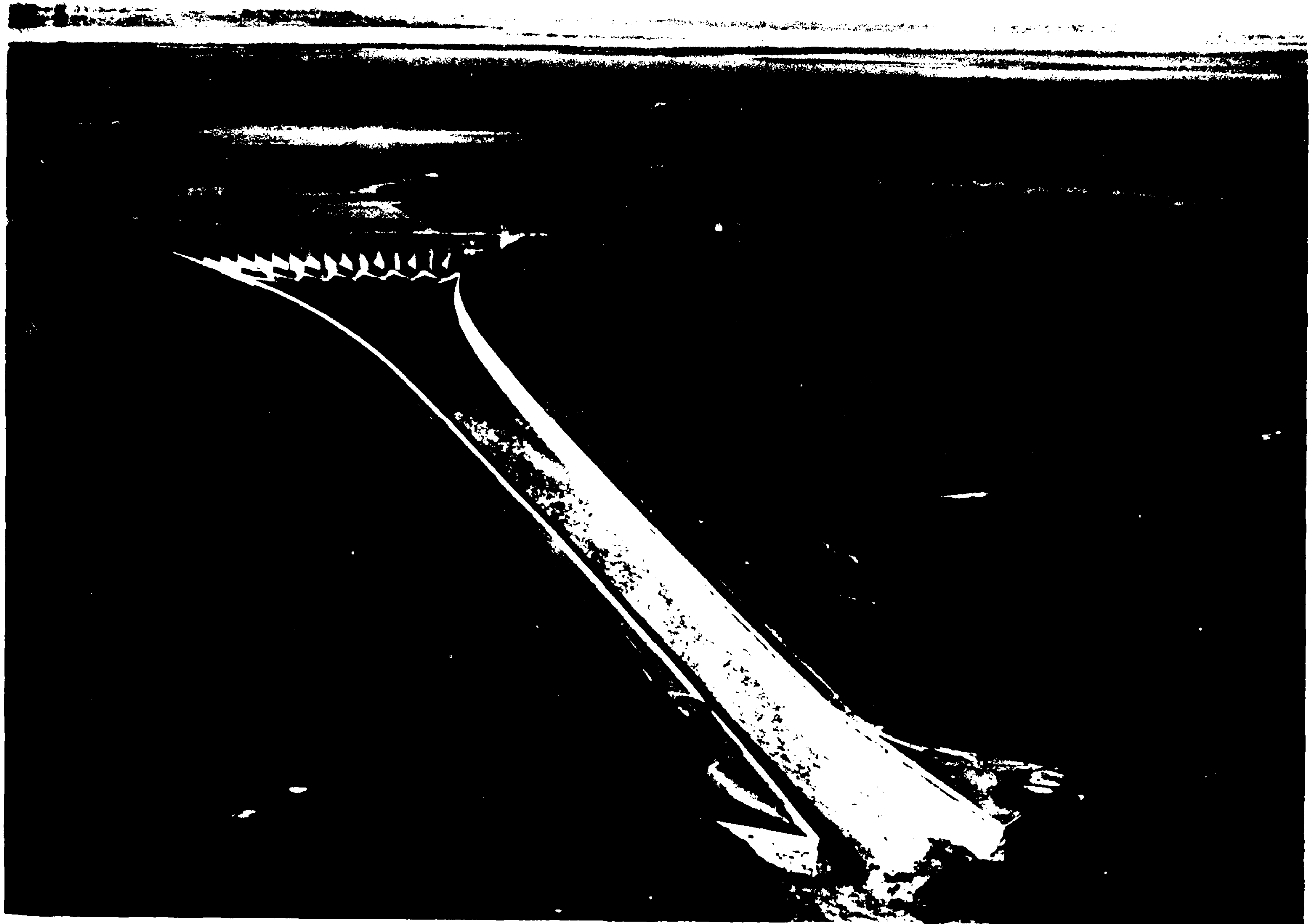

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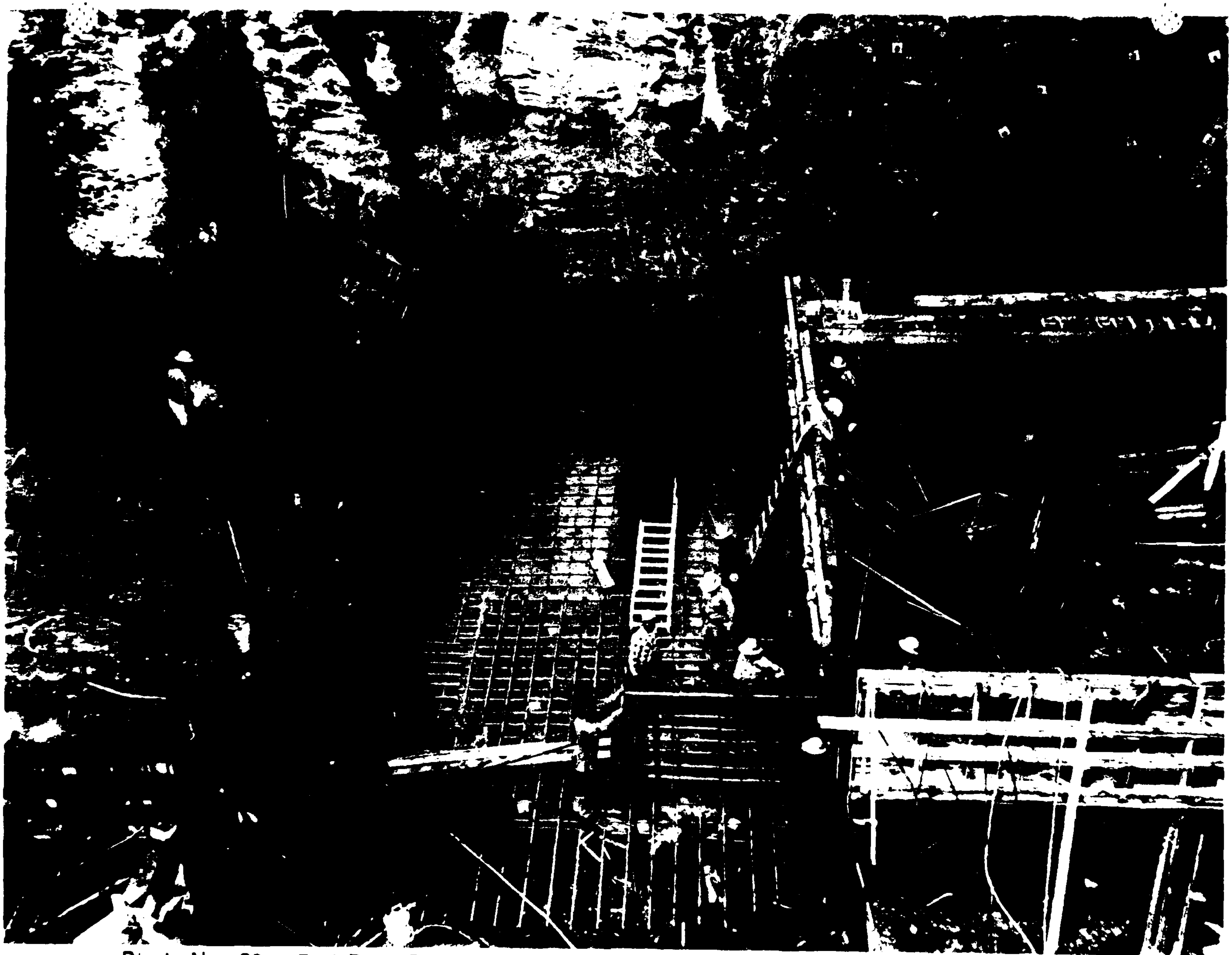

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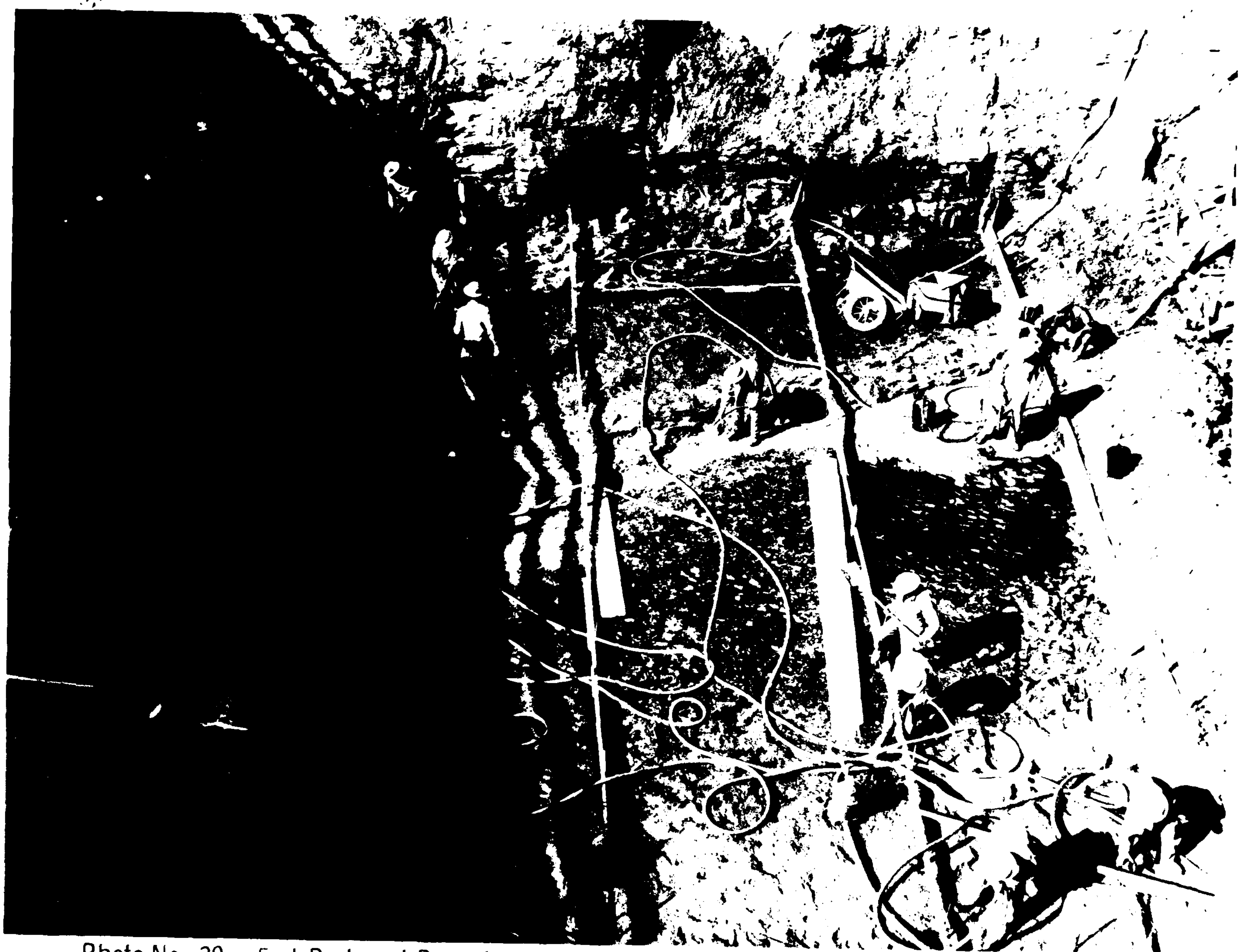

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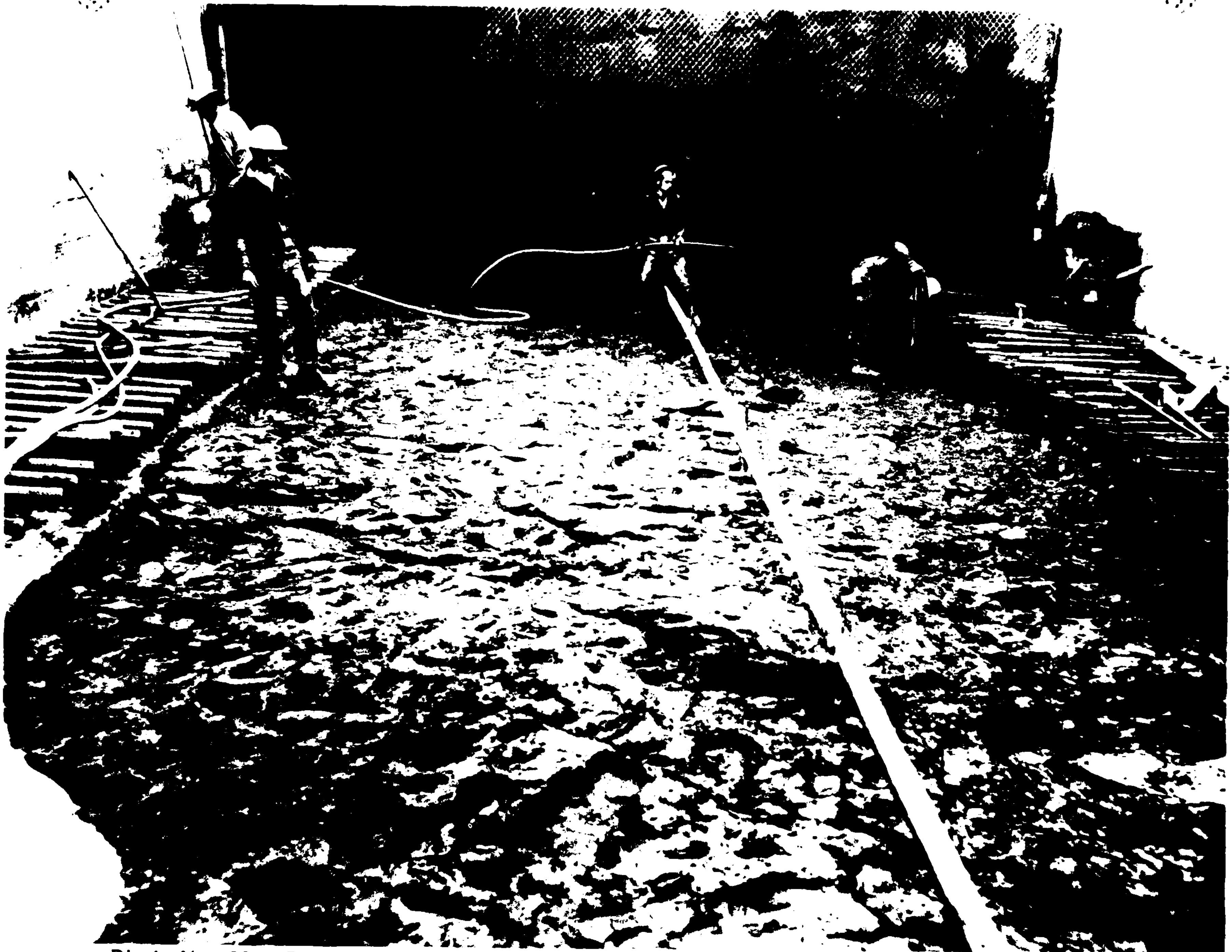

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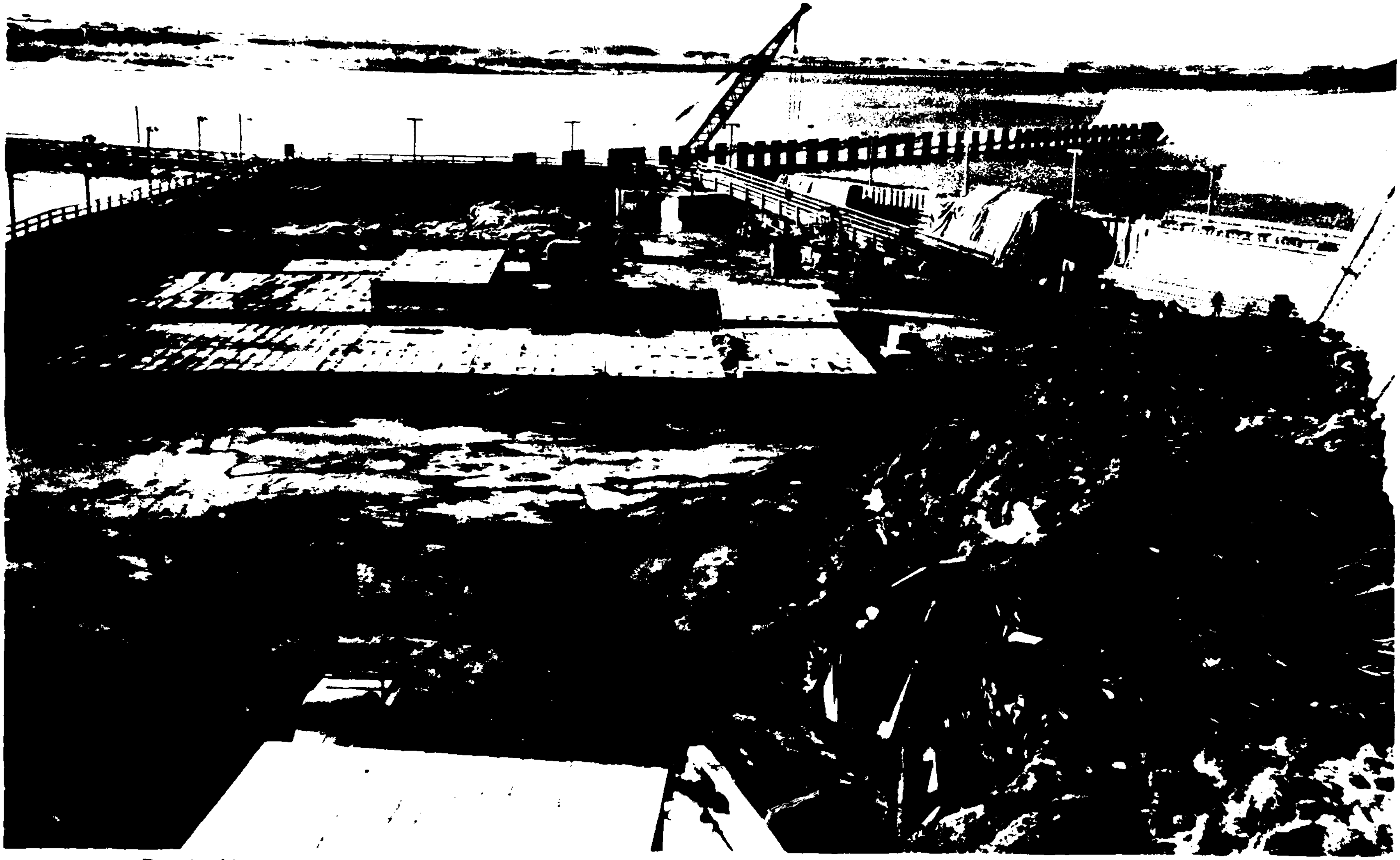

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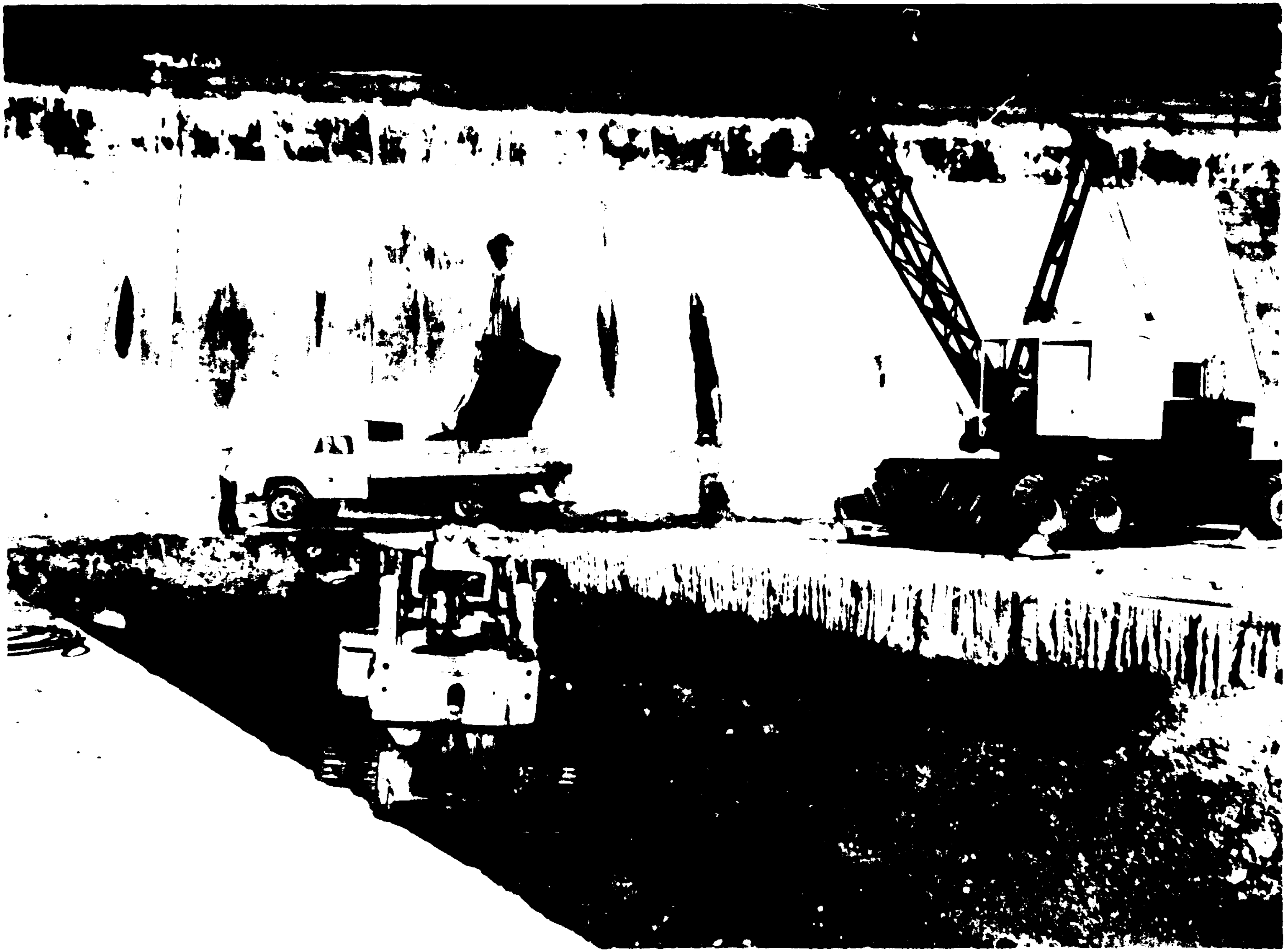

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U.S. ENGINEER OFFICE — WAR DEPARTMENT — FORT PECK, MONTANA

Contractor: Joel Norling
Contract No.: W-24-016-eng-82
Contract Date: 27 April 1946
FUND: 21X3000 M&I of Existing R & H Works

Permanent Relief Well System-Contract Operations — 25 Sept. 1946 #8988

Hydrostatic conditions along D.S. toe - Installation of permanent well casing. View showing wooden casing being installed in well D 64.45/20. 6" wooden casing installed inside temporary 10" steel drill casing which is then removed. Due to buoyancy the wood pipe had to be forced down into place and then weighted down while temporary casing is removed.

Photo No. 35

Photo No. 36

Photo No. 37

FORT PECK FOUNDATION REPORT

CHAPTER 1. - INTRODUCTION

1.1 LOCATION AND GENERAL DESCRIPTION.

1.1.1 Fort Peck dam is located on the Missouri River 1,869 miles (1941 adjusted) above its mouth, in McCone and Valley Counties in northeastern Montana. This location is approximately 19 miles southeast of Glasgow, Montana and about 12 miles south of Nashua, Montana. Plate 1 shows the general project location and plate 1A shows the location of the various features within the project.

1.1.2 The Project is multi-purpose and consists of an earth fill dam, gated concrete spillway, two hydroelectric generating plants, flood control outlet works and a multi-purpose reservoir operated as a unit in the comprehensive plan for flood control, irrigation, navigation, power development and recreation. Table 1 summarizes the overall project.

TABLE 1
FORT PECK DAM AND RESERVOIR
MISSOURI RIVER
MONTANA

PURPOSE

Flood Control, Navigation, Power, and Recreation

LOCATION OF DAM

State	Montana
Counties	McCone and Valley
River	Missouri River, 1771.5 miles above mouth (1960 mileage)
Town	Approximately 19 miles southeast of Glasgow, MT

DRAINAGE AREA

Above Fort Peck Dam, sq. mi.	57,725

STREAMFLOW DATA

Maximum Discharge of Record at Dam Site (1953)	137,000 cfs
Average Annual Runoff at Dam Site, A.F. (1898-1960)	7,000,000 acre-feet

RESERVOIR DATA

Approximate Length of Full Reservoir (in River Miles)	189, ending near Zortman, MT
Shoreline, miles at El. 2234	1,520

	Elevation M.S.L.	Gross Storage Acre-Feet	Gross Area Acres
Maximum Operating Pool	2250	19,400,000	245,000
Maximum Normal Operating Pool	2246	18,400,000	
Minimum Flood Control Pool	2234	15,700,000	
Limit of Drawdown	2160	4,500,000	97,000
Exclusive Flood Control	2246-2250	1,000,000	
Seasonal Flood Control	2234-2246	2,700,000	
Subtotal Flood Control	2234-2250	3,700,000	
Multiple Purpose	2160-2234	11,200,000	
Dead Storage	2030-2160	4,500,000	

DAM

Embankment, Type - Hydraulic and Rolled Earth Fill	
Top of Embankment, Elev. Ft. msl	2280.5 Total Crest
Length, feet	21,026
Maximum Height, feet	250.5
Damming Height, feet (Low Water to Max. Oper. Pool)	220
Top Width, feet	50
Maximum Base Width, feet	3,500
Maximum Base Width w/o Berms, feet	2,700
Fill Quantity, cubic yards	125,628,000

SPILLWAY

Location	Right Bank - Remote
Type - Chute, Concrete Lined with Gated Overflow Weir	
Crest Elevation, feet msl	2,225
Crest Length, Gross, feet	820
Net, feet	640
Gates - Stoney Vertical Lift - No. & Size, feet	16 - 40 x 25
Design Discharge Capacity, cfs	250,000
Discharge Capacity at Maximum Operating Pool, cfs	230,000

OUTLET WORKS

Location	Right Bank
Type - Concrete Lined Tunnels	
Tunnels, No. and Dia. in feet	2 - 24.67
Tunnels Length, feet	6,615 & 7,240
Gates, Type	Cylinder
Gate Size, feet	8.5 x 28 Dia.
Average Discharge Capacity, per Tunnel, cfs	22,500
Total, cfs	45,000
Present Tailwater Elevation, feet msl	2033-2035
Intake Invert Elevation, feet msl	2095

POWER STRUCTURES

Location		Right Bank
Tunnels Type - Concrete Lined with Steel		
Tunnels, Number and Dia. in Feet		2 - 22.33
Tunnels Length, Feet		5,653 & 6,355
Gates, Type		Cylinder
Gates per Tunnel, No. and dia. in feet		1 - 28
Surge Tanks - Number and Dia. in feet	PH #1	3 - 40
	PH #2	2 - 65

POWER INSTALLATION

Average Gross Head Available, feet		205
Number of Generating Units		5
Turbines, Type		Francis
Turbines, Speed, rpm	PH #1	2 - 128.5, 1 - 164
	PH #2	2 - 128.6
Discharge Capacity at Rated Head, cfs	PH #1	7,800
	PH #2	7,200
Generator Rating, kw		2 - 35,000, 1 - 15,000
		2 - 40,000

POWER AVAILABLE

Plant Capacity, kw	165,000
Dependable Capacity, kw	166,000
Average Energy, kwh	979,000,000

1.2 AUTHORITY AND HISTORY OF CONSTRUCTION

1.2.1 Construction of the Fort Peck Project was approved by President Franklin Roosevelt under the Public Works Administration as Project No. 30 on October 14, 1933 and funds were made available on October 24, 1933 as part of the public works program. Photos 36 and 37 show President Roosevelt during several trips he made to the project. The project was formally authorized by the 1935 River and Harbor Act. Installation of power was authorized by Public Law 529, 75th Congress, approved May 18, 1938. Multiple-purpose operation of the Fort Peck Reservoir was authorized by the 1944 Flood Control Act.

1.2.2 Clearing operations began on October 23, 1933. Subsurface explorations commenced August 10, 1933 by the E.J. Longyear Company. Contracts for sand, gravel, and crushing/screening were awarded in December, 1933. Notice to proceed for construction of the tunnels, emergency gate shaft excavation, and main control shafts was issued in June of 1934.

1.2.3 A contract for stripping operations of overburden at the base of the dam was awarded in May 1934 and the operation was completed in November 1934. In July 1934, the embankment cutoff wall contract was awarded and this item of work was accepted by the government in December 1935.

1.2.4 Placement of gravel and rock toes of the dam began in October 1934 and, closure for this phase of work was in July 1937. The placement of dredged fill for the dam was completed in November of 1939, and work on the remaining 25 feet of mechanically placed fill was completed in October 1940.

1.2.5 Power development at Fort Peck was unusual when compared to most projects which include development of power in the initial planning. The steel lining of Tunnel No. 1 from the main control shaft downstream was the only initial planning made for power. The outlet structure was not designed for power development and subsequent planning for power had to be adapted to the existing structure. The planning and construction of Powerhouse No. 1 was quite unusual, due in part to the war which caused wartime restriction on

materials. The restrictions caused the construction of Powerhouse No. 1 to extend over a period of about 12 years and the initial 5 years of operation was without the use of surge tanks. Construction of Powerhouse No. 2 was completed in about 5 years. The following paragraphs give a brief chronological review of the history of planning and construction of the powerhouses.

1.2.6 Powerhouse No. 1. The original plans for powerhouse No. 1 provided for supply contracts for the major items of electrical and mechanical equipment; an initial construction contract for the substructure; and a major construction contract for the superstructure, surge tanks, penstocks, installation of electrical and mechanical equipment, construction of related structures and substation, and remaining work on Tunnel No. 1.

1.2.7 In 1942, the need for additional electrical power for war industries and military installations led to a decision to complete the powerhouse sufficiently to permit the installation and operation of one 35,000 kw generator without surge tanks. The decision required redesign and construction work to be carried out concurrently and additional redesign was necessary at times when substitutions had to be made for critical materials which could not be obtained. Construction work was carried on at an accelerated rate and the 35,000 kw generator was placed in operation 1 July 1943.

1.2.8 Following World War II, plans were made for the completion of the powerhouse structure as originally planned and the addition of the No. 2 15,000 kw Unit.

In 1945 and 1946, alterations to the penstocks and to the main control shaft No. 1 transition section were made which included the removal of the temporary steel penstock to Unit No. 1 and construction of a wye-branch to permit connection of the main penstock to the separate penstock of each of the three ultimately planned units.

1.2.9 A contract was awarded on 16 April 1946 for completion of the powerhouse superstructure and installation of Unit No. 2. Unit No. 2 was initially placed in service for regular power production on 1 February 1948.

1.2.10 A contract was awarded on 3 July 1946 for the construction of the surge tanks, surge tank riser and surge tank housing structure. The surge tanks, including the tank for future Unit No. 3, were completed and placed in service on 17 June 1948.

1.2.11 A contract was awarded on 22 March 1950 for the installation of Unit No. 3 (35,000 kw) with associated equipment and appurtenant work. The installation was essentially complete in December 1951, and Unit No. 3 was placed in initial service.

1.2.12 The contract for the foundation of the powerhouse and for the modification of control shaft No. 2 was awarded on 27 February 1957. The work was completed by 31 December 1958. During September of 1958, a contract was awarded for the fabrication and erection of the penstock liner and the surge tanks. During December of this same year, the contract for construction of the powerhouse superstructure, including installation of machinery and other electrical equipment and the switchyard, was awarded. The tilt of the powerhouse structure during construction caused some delay in this contract in that the embedded machinery had to be reground to level alignment (with the surge tanks full) before the turbines and generators could be installed. Units No. 4 and No. 5 were installed and were placed on a mechanical test run in May 1961.

1.2.13 The chronological order in which the construction of the tunnels and shafts took place is tabulated below.

Work	Started	Finished
Tunnels	May 1934	June 1937
Main Control Shafts	October 1934	November 1936
Emergency Gate Shaft	November 1934	Summer 1936
Steel Lining No. 1	August 1936	June 1940
Emergency Gates	November 1936	June 1937
Intake Reconstruction	August 1939	March 1940

1.2.14 The contract for the unlined portion of the spillway was awarded in September 1934 and work began in November 1934. The open cut excavation of 2,808,000 cubic yards was completed and accepted by the government in October 1935. The notice-to-proceed for the lined channel portion of the spillway was issued in December 1934 and this portion of the spillway was completed in September 1937. Work on the spillway gate structure started in May 1935, and the structures were completed during the 1938 construction season.

1.3 PURPOSE OF REPORT. This report has been prepared in accordance with requirements set forth in Regulation No. 1110-1-1801, Engineering and Design Construction Foundations Reports, Office, Chief of Engineers dated 14 January 1972. The purpose of the report is to compile a record of the foundation conditions encountered for the various structures comprising the Fort Peck dam. Included are pertinent data concerning the site geology, explorations, engineering characteristics of overburden and bedrock, excavation methods, foundation treatment, instrumentation and other related subjects. Also included are supplemental drawings, logs, test data and photographs. Detailed information concerning materials testing, stability analysis, and other foundation design data relating to the project are available from the numerous files of the Omaha District Office and will not be repeated in this report.

1.4 CONTRACTORS. The principal contractors for the various stages of work were:

Contract	Contractors	Award Date
Initial Foundation Investigations.	E.J. Longyear	Aug. 1933
Tunnel Centerlines.	Sam J. Mathews	Dec. 1933
Spillway - Lined Channel.	Massman Construction	Dec. 1934
Spillway - Unlined Channel	Martin Wunderlich	Sep. 1934
Spillway - Gate and Cutoff	Addison Miller, Inc.	Apr. 1935
Structures -	Fielding and Shepley, Inc.	
Powerhouse No. 1		
Initial Design	Harza Engineering	1938
Substructure and Surge Tank Foundations	Woods Bros.	Jul. 1940
Redesign, Modified Power Plant	Harza Engineering	May. 1942

Contract	Contractors	Award Date	
Powerhouse No. 2.			
Preliminary Design	Erik Floor, Assoc.	Oct.	1952
Powerhouse Foundation	E.V. Lane Co.	Feb.	1957
Superstructure and Switchyard	Eagle Construction	Dec.	1958
	Western Construction		
Embankment Stripping Overburden	Addison Miller, Inc.	May.	1934
	Fielding and Shipley, Inc.		
Cutoff Wall	Frazier-Davis Construction	May.	1934
	G. L. Tarlton	Jul.	1934

CHAPTER 2. - FOUNDATION EXPLORATION

2.1 INVESTIGATIONS PRIOR TO CONSTRUCTION: Prior to 1950, approximately 1,018 holes were drilled in relation to foundations at Fort Peck Dam. Core hole locations are shown on plate 2. Core drilling was commenced on site in July of 1933, and by May of 1934 a total of five drilling contracts had been awarded, three of which were for foundations of the main dam. In addition to the above contracts, there was some foundation drilling by Government forces. However, after completion of the above contracts, the major share of the drilling by Government forces was for borrow pit investigations in areas removed from the main dam proper. For this reason, the data obtained from the original subsurface investigations carried out by the above contracts constitute the bulk of information on foundation conditions under Fort Peck dam.

The exploration contracts are summarized below:

Contractor	Purpose of Contract	No. of Holes	Total Feet Drilled
Longyear (1)	Foundation and abutments	42	6098.7
Mathews	Tunnels and Foundation	182	8559.3
Matt	Tunnels, Shafts Portals	30	4716.7
Diamond	Foundation, Borrow, Left Abutment	224	26,300.1
Longyear (2)	Three Spillway Sites	75	11,040.2

2.1.1 Descriptions and evaluations of the three contracts under which most of the foundation investigation for the Fort Peck Dam was accomplished and subsequent drilling by Government plant and hired labor are as follows.

2.1.1.1 Main Embankment.

1. E.J. Longyear Exploration Company (First Contract). The contract was awarded on 18 July and completed on 15 December 1933. The work included the preliminary underground exploration of the dam and dike extension. The main emphasis of the contract was to obtain cores of the shale. There were no provisions for sampling the overburden other than to catch samples from drill fluid (cuttings). Each hole was fishtailed through the overburden to shale bedrock and then the overburden was cased. Information on shale bedrock, under this contract, did not include detailed descriptions but classified the shale as firm, subfirm, or soft and weathered. The main value of this contract was to establish top-of-shale for subsequent work related to the placement of the cut off wall beneath the embankment.

2. Sam J. Mathews Company. The contract was awarded on 3 November, 1933 and completed on 17 March 1934. During most of this contract, overburden samples were taken by means of a churn chopping bit and a bailer. Samples taken by this method were considered misleading so during the latter part of the contract, samples were taken every 2 feet by means of a drive pipe. For the core drilling operation in shale, the drill logs did not show sufficient detail to identify faulting, bentonite marker beds, etc., but mainly classified the shale as firm, subfirm or soft and weathered shale - as in the previous contract.

3. The Diamond Drill Contracting Company. The contract was awarded on 28 March 1934 and completed on 8 September 1934. Drilling for this contract was for final test borings of the dam foundation and proposed borrow areas. Drive samples were specified to be taken at intervals of not more than 10 feet. Equipment consisted of drills that were able to wash the hole clean between drive samples. The method of operation was as follows: Five feet of hole would be fishtailed and a 5 foot length of casing set; A clam shell

sample was then driven into the material until it was certain that 3/4 foot of undisturbed material had been obtained. The hole was bailed between samples and the casing set deeper. The sampling of pervious material by this method was found to be very difficult and often it was not possible to obtain representative undisturbed samples. Only a few samples were recovered to represent the pervious zone. This contract furnished a major portion of the available information on the foundation beneath the Fort Peck Dam.

2.1.1.2 Spillway. Reconnaissance surveys were made on five spillway locations. Preliminary borings were made and test pits sunk at each location. Based on cost and engineering criteria, the present spillway location was selected and a contract let for 71 additional bore holes. The holes varied from 30 to 90 feet in depth. In addition, three test pits, ranging from 24 to 78 feet in depth, were dug along the centerline of the spillway by Government forces. The purpose of the investigation was to establish top of firm shale and to give prospective bidders an idea of the nature of subsurface materials.

2.2 INVESTIGATIONS DURING CONSTRUCTION.

2.2.1 Government Plant and Hired Labor Drilling. The drilling that was done at Fort Peck after the completion of the first five contracts was performed by Government plant and hired labor. Drilling was done by churn drills with long stroke jars and drive barrels. Continuous samples for the entire length of the hole were obtained. Drilling was done in relation to: borrow pit investigation, special abutment investigation, piezometer installation and some pressure relief wells. However, most relief wells were drilled by rotary methods (wash boring). Between 1938 and 1951, 109 holes were drilled into the embankment to obtain samples of the dam's core, consolidation studies, and installation of piezometers and seepage pipes. After a major slide on the right abutment (Sept. 1938), explorations were made in this area, with special attention toward determining physical properties of weathered shale and the bentonites. Holes were drilled to investigate the "A" Fault on the right abutment. Piezometers were installed to determine the possibility that seepage from the reservoir would occur through the abutment

by way of the "A" Fault. These piezometers are shown on plate 61. A foundation study of the left abutment commenced after the slide on the right abutment. Explorations consisted of drilling 29 holes on 300-foot centers. Beginning in 1943, 111 holes were drilled for exploration and piezometer installation in the left abutment area to supplement earlier contract drilling. Piezometers for the main embankment and left abutment are indicated on plate 57.

For other investigations, refer to sections on Powerhouse slope problems, spillway problems and powerhouse problems in Chapter 9.

CHAPTER 3. - GEOLOGY

3.1 PHYSIOGRAPHY. Fort Peck Dam is located in Northeastern Montana between the glaciated and unglaciated portions of the Missouri Plateau section of the Great Plains physiographic province. Eastern Montana is a region of rolling upland plains broken by occasional higher hogbacks and buttes and by dissected, badland topography and flat lowlands along the courses of streams. The exact character of the topography varies from place to place according to the underlying rock formation. The area east of Fort Peck is a broad belt of shale badlands drained by branches of several ravines which are dry most of the year. The western portion of the valley at the site is formed by the end of a broad spur of the upland which is bounded on the north by the valley of Galpin Creek and on the south by several meanders of the river itself.

3.2 TOPOGRAPHY. The site of the dam is located on the Missouri River about 11 miles above the mouth of the Milk River, in Northeastern Montana. The river at this point flows for a short distance in a northerly direction. The elevation of the river bed at the axis of the dam was 2,025 feet (mean sea level), and the average elevation of the flood plain of the valley along the axis was originally 2,055 feet (mean sea level). The river valley, cut into the shale by the action of large water flows during the ice age, has since been built up by river deposits of gravel, sand, silt and clay to a maximum depth of 160 feet. The topography of the east abutment is comparatively rugged, the hills rise 300 feet above the valley floor to an approximate elevation of 2350 feet and are cut by numerous, small valleys and gullies. Bluffs along the west abutment rise abruptly to elevations approximating elevation 2210 after which the adjacent plain gradually increases in elevation.

3.3 REGIONAL GEOLOGY.

3.3.1 Eastern Montana is underlain by a series of late Mesozoic and early Tertiary formations which are found over large areas of the Great Plains. In general, strata dip toward the east at a low angle so that the younger formations are exposed on the east and the older formations to the west. The combination of gentle local warpings with dissection by streams has produced considerable irregularity of outcrop patterns. Most commonly, the outcrops of lower, older formations extend downstream in the valleys for some miles eastward from points where younger formations appear on the bluffs.

3.3.2 Fort Peck is located within the outcrop belt of the Bearpaw Shale, which forms the floor of the Missouri Valley for nearly one hundred miles to the east. However, on the higher parts of the upland, southeast of the dam site, the Lance Formation crops out. Only the Bearpaw Shale is of importance for foundation conditions at Fort Peck.

3.3.3 Prior to glaciation, the Missouri River flowed in a general northeasterly direction from Fort Benton around the north side of the Bearpaw Mountains and then east in the valley now occupied by the Milk River. This preglacial drainage of the Missouri River was dammed by the advancing ice sheet which ultimately diverted the course of the river in an easterly direction south of the Bearpaw Mountains and Little Rocky Mountains into the present course of the Missouri River.

3.3.4 Regionally, thrust faulting has occurred in the area around the Bearpaw Mountains. South of the mountains, the faults are generally confined to a zone, 20 to 30 miles wide. Beyond this zone, strata is relatively undisturbed. Most faults are circumferential to the Bearpaw Mountains and are separated by belts of flat-lying strata that are 1 to 3 miles wide. Deposits of Pleistocene glacial drift cover most of the upland plains areas with the exception of the area south of the Bearpaw Mountains and the adjacent Little Rocky Mountains.

3.4 DESCRIPTION OF OVERBURDEN.

3.4.1 The soil foundation for the main section of the dam consists of a top layer of sandy soil about 25 feet thick. The sandy soil is underlain by approximately 50 feet of clay. Beneath the clay is a pervious soil layer consisting of 40 to 80 feet of sand and gravel with clay lenses. The middle, or clay layer, that overlies the bottom sand and gravel, forms a seal on top of the shale at the left abutment. The clay extends across the valley and feathers into semi-pervious material at the right abutment. The middle clay layer extends about 5,000 feet downstream and approximately 6,000 feet upstream.

3.4.2 The left abutment is covered with a mantle of glacial till which averages about 150 feet in thickness in the area that is adjacent to the main dam. There is a layer of sandy alluvium between shale bedrock and the overlying glacial till. The alluvium varies from a trace to as much as 40 feet in thickness. The typical glacial till, from the ground surface to the sandy alluvium above the shale, consists of a lean clay. The till is almost wholly unstratified and consists of impervious clay with varying amounts of sand, gravel and cobbles. Throughout the left abutment area, the till is typically yellow lean clay from the surface to a depth of approximately 60 feet. At this depth, the color changes to black or gray. There is no discernable difference in the physical characteristics of the soils of the two colors. The sandy alluvium shows definite stratification and varies from a gravelly material directly on top of the shale bedrock to a layer of fat clay directly beneath the glacial till.

3.4.3 The coarse material on top of the shale is a mixture of sand, gravel, shale pebbles, lumps of clay and lignite and is quite pervious. Above the coarse material there is usually a layer of fine to coarse-grained sand. The sand grades upward into a very fine sandy loam which, in places, is quite thick. Capping the pervious strata in most of the left abutment area is a layer of fat alluvial clay which is dark in color. Overburden material with a permeability of 20 x 10-4 cm/sec or greater is defined as pervious, and

materials with permeability range between 1 to 20 x 10^{-4} cm/sec are considered semipervious. Soil profiles beneath the dam are shown by plates 3 through 9, and the relationship of the till to the left abutment is shown by plates 10, 11, and 12.

3.5 BEDROCK STRATIGRAPHY. The Bearpaw Shale is the only bedrock formation of concern for foundation conditions at Fort Peck. The Bearpaw formation consists of dark gray, or black, nearly uniform clay shale beds of marine origin. It is about 1,100 feet thick where the upper portion has not been removed by erosion and is remarkably uniform in appearance from top to bottom. Beds of bentonite occur at different intervals and vary from a few hundredths of a foot to several feet in thickness. Upon exposure, the Bearpaw shale rapidly weathers into an alkaline gumbo soil.

3.6 BEDROCK STRUCTURE.

3.6.1 As previously discussed, the regional dip of the bedrock is very gentle and is essentially horizontal at the project.

3.6.2 The right abutment of the dam was investigated by the original exploration contracts and during tunnel and control shaft construction. After the right abutment slide (discussed in Chapter 9), additional subsurface investigations were conducted in this area. The right abutment contains numerous minor faults and, at the axis of the dam, there are two major faults that are nearly at right angles to the axis and extend some distance upstream and downstream. A fault which received much attention was the "A" Fault (plate 61) which intersects the ground surface of the abutment between Station 6+50 and 8+00. Another fault in the right abutment area is known as the "No. 1 Fault." This fault intersects Shaft No. 1 close to the bottom of the shaft and also intersects Shafts 2 and 3 at higher elevations. Fault patterns in the right abutment area are shown on plate 36.

3.6.3 No evidence of recent movement for either fault was indicated by field investigations. During tunnel excavation, dowel movement points were installed across several major faults but no movement was detected.

3.6.4 It was observed that gouged areas within fault planes served as seepage paths for water. Exploratory holes were drilled and piezometers installed to investigate the "A" Fault to determine if there was any possibility of seepage from the reservoir making its way through the abutment by way of this fault. Six piezometers were installed in the fault zone to determine if there was any possibility that appreciable amounts of seepage could occur along the "A" Fault. In general, it was found in this investigation that the gouge material in the fault zone had weathered enough so that it produced a mud, and it was almost impossible to keep the piezometers open through the faulted zone. A clay mud filled in the hole (uncased) in the area through the fault zone. From this, it was concluded that mud in the fault zone would seal off any open or fractured zones and there was very little chance of appreciable seepage developing through the "A" Fault zone.

3.6.5 Shale bedrock at the left abutment is covered by an average thickness of 150 feet of glacial till. The bedrock surface is almost level and is at an average elevation of approximately 2,060 msl. Shale bedrock at the left abutment continues on the level from the edge of the abutment to a point approximately 8,000 feet to the west. At this point, the top of shale rises abruptly 60 feet in elevation and then continues at a slope equal to the slope of surface topography. Bedrock contours are shown by plate 42.

3.6.6 Perhaps the most important subsurface feature of the left abutment is a so-named "island" of clay shale that is located along the edge of the abutment and downstream from the dike. The bedrock "island" is capped directly with typical glacial till with no intervening layer of pervious material. The "island" of impervious shale capped by impervious glacial till provides a positive cutoff for seepage along the edge of the abutment from a point that almost ties into the core of the main dam to a point well downstream from the toe of the dam. Plate 42 is a plan of the left abutment which shows the high bedrock shale area. In addition, Plate 42 indicates a fault line with a pronounced change in shale elevation within a short distance to the east. The bedrock high and fault line discussed above,

results in left abutment seepage being funneled into a limited area along the abutment (from approximately range 23+00-D to approximately range 37+00-D). Piezometers downstream from range 37+00-D have indicated no changes in hydrostatic pressure.

3.7 BEDROCK WEATHERING. One of the characteristics of the Bearpaw shale is that it dries out rapidly on exposure to air. The drying out produces cracking and with continued weathering by air and water, the shale disintegrates to a gumbo soil. Between the weathered shale and unweathered shale is an intermediate zone that in the past has been called subfirm shale. The depth of firm shale varies considerably. Weathered shale at the ground surface consists of an alkaline gumbo soil which shrinks and cracks on drying out. Below, the material is composed of small, thin fragments which changes gradually to material with the appearance of shale but is soft and can be excavated as a soil. At places the material is reddish or brown in appearance due to the presence of iron oxide. The oxidized shale is also found in the so-called subfirm shale zone. It is probable that the iron oxide color comes from oxidized iron pyrite which is abundant in the Bearpaw Shale.

3.8 GROUNDWATER. In the Bearpaw Formation, ground water movement is controlled by fractures in the shale. Ground water can move vertically or horizontally and at various rates according to fracture or fault patterns in any specific area.

3.8.1 Ground water did not cause serious problems with excavations into bedrock. Water in quantities that required pumping was encountered in the cut-off wall at the lower tunnel portals and originated by seepage along fault and bedding planes into the excavation. The excavation was also below the river elevation. Water in sufficient quantities to drip was found in two places in No. 1 tunnel, in one place in No. 2 tunnel and in one place in No. 2 shaft. The source of the moisture was surface rain and snow melt which migrated along faults and fractures.

3.8.2 No construction problems related to ground water in the Bearpaw Shale occurred for the foundations for the powerhouse or spillway. No problems related to ground water were encountered along the centerline of the main embankment since the foundation cut-off for this portion of the project consists of a sheet pile wall emplaced by water jetting.

3.9 ENGINEERING CHARACTERISTICS OF OVERBURDEN. Overburden beneath the main embankment consists of river silts, gravels, sands and clays. The soil foundation for the main section of the embankment consists of a top stratum of 25 feet of sandy material that is underlain by approximately 50 feet of clay. Underlying the clay is a pervious stratum of sand and gravel 40 to 80 feet thick with clay lenses. The clay over the pervious material acts as a seal against the shale on the left abutment, extends across the valley and feathers into semipervious material at the right abutment. Laboratory test results for foundation surface clays are shown on Plates 29, 30 and 31. Surface clay sample particle sizes range in size from .0035 mm to .08 mm with cohesion values between 0.30 to 0.40 tons per square foot. Phi ($\emptyset$) values are between 2°55' and 11°50' as determined by the formula:

$$\tan \emptyset = \frac{\text{Shearing Resistance - Cohesion}}{\text{Vertical Load}}$$

The surface clays experienced failure in shear tests at applied pressures between 0.325 to 0.44 tons per square foot. The greatest amount of consolidation occurred within the first 64 minutes of testing when 72 percent of consolidation occurred under 1 ton of pressure per square foot. Engineering properties of coarser alluvial material that was used for the embankment shell are shown on plates 34B and 34C. The material has a specific gravity which varies between 2.68 and 2.73 with effective grain sizes between 0.11 and 0.21. Void ratios (remoulded samples) for embankment shell material decrease by 0.06 per 4 cc change in volume at failure for lateral shear tests which used a pressure of 1 ton per square foot.

Overburden on the right abutment consists of weathered shale that was a major contributing factor for a large slide which occurred in the area in September 1938. Laboratory tests on weathered shale are presented by Plates 26 and 28. Laboratory tests on weathered shale indicate cohesion values between 0.5 ton per square foot and 0.0 ton per square foot. Values of ϕ for the samples tested are 15°25' and 16°23'. Shear tests indicate that the average movement was 0.0074 inches before failure at an average pressure of 1.5 tons per square foot.

Overburden on the left abutment consists of glacial till on top of sandy alluvium which, in turn, overlies shale bedrock.

Above the impervious shale bedrock, the pervious alluvial material was subdivided according to relative permeability. Based on permeability tests run on representative samples, it was considered that a sandy material with a narrow size range, with a median sand size of approximately 0.15 mm, and clay content of 5 to 6 percent, would mark the dividing line between pervious and semipervious materials. Materials shown as semipervious on Plates 10, 11 and 12, probably range from 5 to 15 x 10^{-4} cm/sec and the pervious material from 25 to 400 x 10^{-4} cm/sec.

3.10 ENGINEERING CHARACTERISTICS OF BEDROCK. The following description of the engineering characteristics of the Bearpaw Shale is taken from descriptions of the shale in the vicinity of the spillway.

Weathered shale (Photo 2) is composed of small, thin fragments with gypsum and alkaline salts and is easily excavated. Oxidized shale (iron oxide) is reddish or brown in appearance with cemented joint planes which result in varied degrees of hardness. During construction, the material was found to be often badly jointed but normally could be easily excavated. Subfirm shale is bluish in color and badly jointed with irregular fractures. Moisture content is usually above 16% by weight, and open joints are usually

filled with selenite. Firm shale is dark blue gray, dense and has a specific gravity of 2.25. Joint planes are tight and selenite is absent. Moisture content varies between 14 and 16% by weight.

Bentonite occurs throughout the Bearpaw Shale. Certain types of bentonite have a strong affinity for moisture which results in large increases in volume. When exposed to air, bentonite disintegrates to a powder. When wet it is a slippery mass that resembles soft soap. During construction, various shale slopes were studied, and it was learned that the shale would not stand without bracing on slopes steeper than 1 on 1. (Photo No. 12)

The following information on more detailed engineering properties of the Bear Paw Shale is from laboratory tests that were run on samples of the shale obtained by drilling in the spillway area in 1965. A great number of tests were made to determine as many of the physical characteristics of the shale and bentonite as possible. Bentonite samples were very limited. One of the main purposes of the testing program was to determine the dynamic reaction of the shale over periods of time with controlled values of stress. The summary and discussion in the following paragraphs, cover the main characteristics of the shale and the test results are included as tables. Moisture samples were taken at 2-foot intervals for the entire depth of each drill hole. The moisture content was higher at the surface and decreased to about the 60-foot depth, then remained constant to 180-foot in depth and then again decreased. The wet density and dry density vs. depths are also shown for each drill hole. There is also included void ratio and percent saturation plots vs. depth. The void ratio is variable but does indicate a lower average void ratio with depth.

3.10.1 Direct shear tests were run to determine the stress vs. the rate of strain for representative samples of shale and bentonite. ıis data is summarized by table 2 and represents a great number of individual incremental tests, each of which took 24 to 72 hours to perform. It should be noted that a rate of strain of 1×10^{-7} inches per minute represents just over one-ten thousandth inch of movement for each 24 hours. The rate of strain values represents the steady rate of strain after a period of time, usually 24 hours or longer.

Table 2

Direct Shear (Initial Shear)
Tabulation of Shear Values
Sample U-9

Rate of Strain	Tan Ø	Cohesion ton/sq. ft.
1×10^{-6} inches per min.	0.60	1.50
1×10^{-7} inches per min.	0.42	1.40
1×10^{-8} inches per min.	0.32	1.30
Laboratory Report	0.60	1.85

Direct Shear (Initial Shear)
Tabulation of Shear Values
Sample U-23

Rate of Strain	Tan Ø	Cohesion ton/sq. ft.
1×10^{-6}	1.17	1.10
1×10^{-7}	0.90	0.70
1×10^{-8}	0.80	0.40
Laboratory Report	1.28	0.10

Table 2 (Cont'd)

Sample U-20 (Bentonite)

Rate of Strain	Tan Ø	Cohesion ton/sq. ft.
1 x 10-5	0.70	0
1 x 10-6	0.48	0
1 x 10-7	0.33	0
1 x 10-8	0.23	0
Laboratory Report	0.67	0

3.10.2 Friction Tests. The friction tests were designed to obtain true friction data which could be applied to weakness planes such as joint planes and faults where two separate shale surfaces are in contact and movement between the surfaces would constitute frictional resistance.

The summary results of the friction testing program are shown on Table 3.

Table 3

Fort Peck Spillway
Tabulation of Shear Values for Bearpaw Shale
for Different Rates of Strain, for Prepared Shale Surfaces
of Direct Shear Reshear Tests

Rate of Strain	Friction Values Tan Ø	Cohesion ton/sq. ft.
Wet Surfaces		
1×10^{-2} in./min.	.21	0.20
1×10^{-3} in./min.	.17	0
1×10^{-4} in./min.	.12	0
1×10^{-5} in./min.	.52	0.90
1×10^{-6} in./min.	.45	0.30
1×10^{-7} in./min.	.35	0
Dry Surfaces		
1×10^{-2} in./min.	.25	0.30
1×10^{-3} in./min.	.25	0.10
1×10^{-4} in./min.	.24	0
1×10^{-5} in./min.	.87	0
1×10^{-6} in./min.	.59	0
1×10^{-7} in./min.	.45	0

3.10.3 Unconfined compression tests. The unconfined compression tests were run with increments of constant stress and the rate of strain was observed for each test. The time to failure ranged from 6 to 15 days, except the initial test which was failed in two days to obtain an idea of the strength of the material. The tests were all run at the natural moisture content. Test results are shown by table 4.

Table 4

Fort Peck Spillway
Unconfined Compression
Tabulation of Data

		U-14	U-27	U-4	U-23	U-19	U-7	U-16	U-16	U-16
Water Content	Wo	15.1%	14.5%	17.4%	14.0%	13.1%	15.8%	15.1%	15.2%	15.2%
*Void Ratio	Co	.431	.404	.484	.397	.368	.451	.416	.425	.425
*Saturation	So	96.2	98.6	98.9	97.2	97.8	96.3	100	98.7	98.7
Density Dry	Vd	119.9	122.2	115.6	122.8	125.4	118.3	121.1	120.5	120.5
Density Wet		138.0	139.9	135.7	140.0	141.9	137.0	139.9	139.0	138.9
Time Failure in days	Te	6	9	14	2	6	6	7	15	8
Depth in feet		98	211	32	179	152	55	128	128	128
Liquid Limit	LL	109	127	104	128	110	125	102	102	102
Plastic Limit	PL	27	20	26	21	23	26	28	28	28
Plastic Index	PI	82	107	78	107	87	99	74	74	74
Specific Gravity	Gs	2.69	2.72	2.70	2.71	2.80	2.84	2.75	2.75	2.75
Ht/Dia. Ratio		2-1	2-1	2-1	2-1	2-1	2-1	2-1	1-1	1/2-1
Failure Tons/Sq.Ft.		55.5	62.2	27.4	55.0	51.7	47.7	55.2	68.7	68.6
Inst. Mod. of Deform. Tons/Sq. Ft.	Mi		12,200	11.900	12,660	12,160	9,615	13.330	9,100	7,110
% Strain to Failure		.74	.75	1.17	.74	.72	.59	.57	.57	.80

*Void ratio and percent saturation computed for an average specific gravity of 2.75.

3.10.4 Swell Tests. The swell potential of the Bearpaw shale has been appreciated since the Fort Peck Project was initiated but very little specific data on swell capabilities of the Bearpaw shale at Fort Peck was available up to 1965. Swell tests were run to determine the swell potential of the shale when restrained laterally, with water available to both top and bottom of sample, under known vertical stresses. It was believed that the dissolved salt content of the natural moisture in the shale had a considerable effect on the swell potential, and the tests touched on this complex problem in the most preliminary manner. Also, the affinity of the shale for water increased as the saturation of the shale decreased due to drying. The adjusted data indicates the average percent swell for a one-inch high sample, with access to distilled water at both ends, for various restraining normal loads. The data indicated it would take about 21 tons per sq. ft. to restrain the one-inch sample from swelling for a 500-hour period. The consolidation test was run on the sample at the as-received moisture content and the sample was not allowed access to moisture during testing. The sample was from 178.3 to 180.3 feet depth. The overburden pressure used for this depth was based on an average wet density of 140 lbs. per cubic ft, and was calculated to be 12.5 tons per sq. ft. Each consolidation load was for 24 hours except for a two ton per sq. ft. load which was for 93-1/2 hours. The total consolidation was 1.03% for 12-1/2 tons per sq. load.

3.10.5 Tensile Tests. A number of tensile tests were conducted on four samples of Bearpaw shale. Three or four specimens from each sample were tested at a constant rate of load application of 600 pounds per minute. A total of 14 samples were quick tested to failure immediately after preparation. The average tensile strength of the 14 samples was 5.9 tons per sq. ft. The average value for each sample ranged from 5.3 to 6.7. The real significance of the tensile testing program was the high strength of the shale in tension and also the apparent increase in strength with continued strain.

3.10.6 Triaxial Testing. Triaxial testing was all performed under increments of constant stress and rate of strain observations were made for each stress increment. The failure stress varied from 47.5 tons per sq. ft. for 4 tons per sq. ft. minor principal stress to 82.5 tons per sq. ft. for 12.5 tons per sq. ft. minor principal stress.

3.10.7 Dissolved Salts. Tests were made to determine the amount of soluble salts present in shale samples and the effect of these salts on some of the characteristics of the shale. Test results are presented in Table 5.

Table 5

Tabulation of Test Data
Soluble Salts Determination
Atterberg Limits

Test Series	Hole No.	Sample	LL	PI	Specific Gravity	Soluble Salts
2	65-1T	U-9	117	91	2.76	2.1%
2	65-1T	U-18	162	135	2.80	3.0%
15	65-1T	U-6	122	96	2.74	0.5%
15	65-1T	U-10	116	89	2.74	0.9%
15	65-2T	U-8	109	81	2.74	0.7%
		Test Data After Removal of Soluble Salts				
2	65-1T	U-9	112	85	2.74	--
2	65-1T	U-18	114	86	2.77	--
15	65-1T	U-6	141	114	2.72	--
15	65-1T	U-10	134	106	2.71	--
15	65-2T	U-8	121	95	2.72	--

3.10.8 Atterberg limits and specific gravity determination: Atterberg limits and specific gravity data are presented below.

Table 6

Test	Test Hole	Atterberg Limits Sample	LL	PL	PI	Specific Gravity
D. Salts	65-1T	U-9	117	26	91	2.76
D. Salts	65-1T	U-18	162	27	135	2.80
D. Salts	65-1T	U-6	122	26	96	2.74
D. Salts	65-1T	U-10	116	27	89	2.74
D. Salts	65-1T	U-8	109	28	81	2.74
Tensile	65-1T	U-18	126	26	100	2.76
Consol.	65-1T	U-23	128	21	107	2.71
Swell	65-2T	U-23	131	22	115	2.74
Swell	65-1T	U-13	108	27	81	2.74
Swell	65-2T	U-15	117	26	91	2.72
Triaxial	65-2T	U-16	117	26	91	2.75
Triaxial	65-1T	U-14	106	28	78	2.75
U. Comp	65-1T	U-27	127	20	107	2.72
U. Comp	65-2T	U-14	109	27	82	2.69
U. Comp	65-2T	U-4	104	26	78	2.70
U. Comp	65-1T	U-16	102	28	74	2.75
U. Comp	65-1T	U-23	128	21	107	2.71
U. Comp	65-1T	U-19	110	23	87	2.80
U. Comp	65-1T	U-7	125	26	99	2.84
D. Shear	65-1T	U-24	144	22	122	2.73
D. Shear	65-1T	U-9	117	26	91	2.67
D. Shear	65-1T	U-10	116	27	89	2.74
Friction	65-1T	U-11	117	26	91	2.66
Friction	65-1T	U-17	118	27	91	2.75
Average Value			120	25	95	2.74

3.10.9 Direct shear tests were run on undisturbed shale, undisturbed bentonite, and pre-cut samples of shale. Reshear tests were run on all undisturbed shale and bentonite samples after initial failure. The pre-cut shale surfaces and the reshear tests were made to simulate the movement of shale on joint planes and fault planes where movement surfaces exist. The greater share of direct shear tests were stress controlled but a number of strain controlled tests were also run to complete the range of movement from 1×10^{-7} to 1×10^{-2} inches per minute. This covers a rate of movement from .004 ft. per year to 438 feet per year. The following tabulation for the initial shear for the 4-ton per sq. ft. normal load shows the rate of strain, amount of strain for each stress value, and the total amount of creep-strain to failure.

Table 7
Tabulation of Movement T.S.F. Normal

Stress Rates in T.S.F.	Rates of Strain in inches/min.	Strain Time in Minutes	Creep Movement in Inches
0.6	6.16×10^{-9}	1440	$.89 \times 10^{-5}$
0.9	4.14×10^{-8}	1440	5.97×10^{-5}
1.2	1.60×10^{-7}	2880	46.14×10^{-5}
1.7	8.23×10^{-7}	4320	355.72×10^{-5}
2.1	2.22×10^{-6}	1680	373.46×10^{-5}
Total Creep Movement			782.18×10^{-5}
		or	.0078"

3.10.10 Miscellaneous Bentonite Controlled Stress Direct Shear Tests. Samples of bentonite were saved for testing of all seams which were about one inch or more in thickness. The samples were all tested as controlled stress direct shear tests at 4 tons per square foot normal load. The following tabulations give the results of this testing.

Table 8
Direct Shear Tests on Bentonite
Controlled Stress at 4 T.S.F. Normal Load

Drill Hole	Sample No.	Depth	Moisture	Density	L.L.	Initial Shear (T.S.F.)
Test Trench						
40+70	U-7	-	55.1	69.6	255	1.6
68-3T	U-8	132.2	26.8	100.4	243	4.1
68-3T	U-16	186.9	24.2	106.1	343	4.4
68-4T	U-7	98.1	12.6	125.5	130	3.9
68-4T	U-12	-	23.4	102.6	278	2.7
68-4T	U-14	165.2	23.9	103.9	268	2.4
68-4T	U-16	-	15.9	115.2	332	2.2
68-5T	U-4	70.5	25.6	101.8	363	3.7
68-5T	U-5	78.2	25.9	101.3	300	2.9

All the bentonites except those from the test trench sheared with such irregular surfaces that no reshear tests could be run. The undisturbed bentonite tested at about the same strength as samples of firm shale.

3.10.11 Additional Direct Shear Testing. Additional controlled stress and controlled strain direct shear tests on the bentonite were conducted on samples from a test trench in the spillway area. For this series of tests, controlled stress and controlled strain tests were run on the material. The following tabulation gives the basic results of these tests:

Table 9
Tabulation of Bentonite
Shear Parameters

Sample	Moist-ture	L.L.	Controlled Strain: Initial Shear		Controlled Strain: Reshear		Controlled Shear: Initial Shear		Controlled Shear: Reshear	
U-7	52.8	255	ϕ=12°	C=.36	ϕ=14°	C=.20	ϕ=19°	C=.31	ϕ=20°	C=.10
U-9	43.2	226	ϕ=23°	C=.10	ϕ=13°	C=.15				
	52.4									

The results of both controlled stress and controlled strain tests for sample U-7 indicated the reshear values were higher strength than the initial shear. This was contrary to most tests, but was taken to mean that at this high moisture content the bentonite renews bonds between particles. The bentonite used for this test had in the past undergone about one foot of movement since the spillway channel excavation was completed.

3.10.12 Unconfined Compression Tests. The unconfined compression tests were run with increments of constant stress, and the rate of strain was observed for each increment of stress. Unconfined compression test data are summarized below by Table 10.

Table 10

Fort Peck Spillway
Tabulation of Data
Unconfined Compression Tests

Sample #	Drill H.	Failure T/Sq.Ft.	Ht. Dia. Ratio	Moist %	Void Ratio	Dry Density	Time of Failure in Day	% Strain to Failure	Inst. Mod. of Def. T/Sq. Ft.
U-14	65-2T	55.5	2-1	15.1	.431	119.9	6	.74	—
U-27	65-1T	62.2	2-1	14.5	.404	122.2	9	.75	12,200
U-4	65-2T	27.4	2-1	17.4	.484	115.6	14	1.17	11,900
U-23	65-2T	55.0	2-1	14.0	.397	122.8	2	.74	12,600
U-19	65-1T	51.7	2-1	13.1	.368	125.4	6	.72	12,160
U-7	65-1T	4[illegible].7	2-1	15.8	.451	118.3	6	.59	9,615
U-16	65-1T	55.2	2-1	15.1	.416	121.1	7	.57	13,320
U-16	65-1T	68.7	1-1	15.2	.425	120.5	15	.57	9,100
U-16	65-1T	68.6	1/2-1	15.2	.425	120.5	8	.80	7,110
U-1	68-3T	24.8	2-1	17.2	0.45	116.9	2	1.30	5,350
U-3	68-3T	74.2	2-1	13.7	0.40	124.1	25	1.05	20,640
U-6	68-3T	33.5	2-1	12.7	0.35	124.7	27	1.33	16,000
U-11	68-3T	34.8	2-1	15.9	0.44	117.7	23	0.85	15,800
U-18	68-3T	*88.2	2-1	14.1	0.36	122.7	*	0.72	--

3.10.13 Dissolved Salts Determination. Tests were made on a number of shale and bentonite samples to determine the amount of soluble salts present. The results of tests made to date are shown in the following table (Table 11). After the samples of the clear liquid were taken and analyzed, the rest of the liquid was decanted off and the leached soil sample was again tested for Atterberg limits and specific gravity. Test results indicate that soluble salts in the pore water of a sample may have a considerable effect on the Atterberg limits of a soil sample. The Atterberg limits determination before and after leaching are as follows, Table 12.

Table 11

Fort Peck Spillway
Soluble Salt Determination
of Shale and Bentonite Samples

Drill Hole Location	Sample No.	% Dis. Salt	% SO_4	% Cl	% Na	% Mg	% Ca	% HCO_3	% NO_3	Remarks
68-3T	U-1	0.806	0.387	0.010	0.262	0.002	0.006	0.234	0.000	
68-3T	U-15	1.471	0.958	0.008	0.328	0.007	0.037	0.013	0.000	
T.T. 40+70	U-9	0.689	0.436	0.003	0.230	0.001	0.002	--	--	
T.T. 40+70	U-9	1.844	1.028	0.010	0.572	0.002	0.006	--	--	Upper Shale
T.T. 40+70	U-9	0.819	0.500	0.005	0.267	0.002	0.005	--	--	Bentonite
T.T. 40+70	U-7	1.514	0.997	0.005	0.462	0.004	0.020	--	--	Lower Shale
T.T. 40+70	U-7	2.742	1.650	0.019	0.909	0.003	0.024	--	--	Bentonite
T.T. 40+70	U-7	0.708	0.372	0.003	0.234	0.001	0.002	--	--	Lower Shale
68-6T	U-1	1.360	0.260	0.050	0.310	0.009	0.021	0.249	0.002	
68-6T	U-3	0.830	0.140	0.006	0.320	0.010	0.005	0.427	0.000	
65-1T	U-9	2.100								
65-1T	U-18	3.000								

Table 12

Soluble Salts Determination Atterberg Limits
and Sp. Gravity

Location and Sample		Material	% Moisture	Before Leaching				After Leaching		
				% Dis. Salt	L.L.	P.L.	Sp. Grav.	L.L.	P.L.	Sp. Grav.
65-1T	U-9	Shale	15.9	2.1	117	26	2.76	112	27	2.74
65-1T	U-18	Shale	15.5	3.0	162	27	2.80	114	28	2.77
68-3T	U-1	Shale	17.2	0.8	121	22	2.72	80	23	2.78
68-3T	U-15	Shale	--	1.5	116	25	2.78	69	26	2.71
68-6T	U-1	Shale	17.5	1.36	147	23	2.65	--	--	--
68-6T	U-3	Shale	15.7	0.86	125	25	2.69	--	--	--
T.T.	U-9	U. Shale	16.7	0.69	123	31	2.75	91	25	2.85
T.T.	U-9	Bentonite	48.0	1.84	226	40	2.97	167	38	3.09
T.T.	U-9	L. Shale	14.7	0.82	106	24	2.74	81	24	2.74
T.T.	U-7	U. Shale	27.3	1.51	143	30	2.78	83	28	2.83
T.T.	U-7	Bentonite	52.8	2.74	255	42	2.88	151	45	2.82
T.T.	U-7	L. Shale	6.6	0.71	112	27	2.64	80	26	2.71

3.10.14 Atterberg Limits and Specific Gravity: The following tabulation gives the results of all Atterberg limit and specific gravity determinations for all Fort Peck Spillway samples tested.

Table 13

Fort Peck Spillway
Atterberg Limits and Specific Gravity

Bearpaw Shale Samples

Test or Remarks	Test Hole	Sample	Moist	L.L.	P.L.	P.I.	Specific Gravity
D. Salts	65-1T	U-9	15.9	117	26	91	2.76
D. Salts	65-1T	U-18	15.5	162	27	135	2.80
D. Salts	65-1T	U-6		122	26	96	2.74
D. Salts	65-1T	U-10	15.6	116	27	89	2.74
D. Salts	65-1T	U-8		109	28	81	2.74
Tensile	65-1T	U-18	15.5	126	26	100	2.76
Consol.	65-1T	U-23	4.5	128	21	107	2.71
Swell	65-2T	U-23	14.8	131	22	115	2.74
Swell	65-1T	U-13	15.2	108	27	81	2.74
Swell	65-2T	U-15	14.3	117	26	19	2.72
Triaxial	65-2T	U-16	14.0	117	26	91	2.75
Triaxial	65-1T	U-14	15.5	106	28	78	2.75
U. Comp.	65-1T	U-27	14.5	127	20	107	2.72
U. Comp.	65-2T	U-14	15.1	109	27	82	2.69
U. Comp.	65-2T	U-4	17.4	104	26	78	2.70

Test or Remarks	Test Hole	Sample	Moist	L.L.	P.L.	P.I.	Specific Gravity
U. Comp.	65-1T	U-16	15.1	102	28	74	2.75
U. Comp.	65-1T	U-23	14.0	128	21	107	2.71
U. Comp.	65-1T	U-19	13.1	110	23	87	2.80
U. Comp.	65-1T	U-7	15.8	125	26	99	2.84
D. Shear	65-1T	U-24	14.1	144	22	122	2.73
D. Shear	65-1T	U-9	16.4	117	26	91	2.67
D. Shear	65-1T	U-10	14.8	116	27	89	2.74
Friction	65-1T	U-11	14.9	117	26	91	2.66
Friction	65-1T	U-17	14.7	118	27	91	2.75
U. Comp.	68-3T	U-1	17.2	121	22	99	2.72
U. Comp.	68-3T	U-3	13.7	102	20	82	2.78
U. Comp.	68-3T	U-6	12.7	92	22	70	2.72
U. Comp.	68-3T	U-11	15.9	140	24	116	2.72
U. Comp.	68-3T	U-18	14.1	112	24	88	2.68
Upper Shale	*	U-1		102	29	73	—
Lower Shale		U-1		83	24	59	—
Upper Shale		U-7	27.3	143	30	113	2.78
Lower Shale		U-7	6.6	112	27	85	2.64
Upper Shale		U-9	16.7	123	31	92	2.75
Lower Shale		U-9	14.7	106	24	82	2.74
Swell Test	68-6T	U-1	17.5	147	23	124	2.65
Swell Test	68-6T	U-3	15.7	125	25	100	2.69
Unconfined	69-3T	U-1		87	22	65	2.75
Unconfined	68-4T	U-3	16.8	110	23	87	2.70
Unconfined	68-4T	U-6	15.5	121	26	95	2.77

Test or Remarks	Test Hole	Sample	Moist	L.L.	P.L.	P.I.	Specific Gravity
Unconfined	68-4T	U-13		111	26	85	2.72
Unconfined	68-4T	U-1		116	24	92	2.77
Upper Shale	68-5T	U-5	17.7	227	35	192	2.78
Lower Shale	68-5T	U-5	16.0	128	26	102	2.77
Upper Shale	68-4T	U-16	14.7	118	24	94	2.77
Upper Shale	68-4T	U-14	15.3	121	26	95	2.70
Lower Shale	68-4T	U-14	15.4	97	27	70	2.68
* Test trench Sta. 40+70 on right berm							
Upper shale	68-4T	U-12	15.1	138	27	111	2.70
Lower Shale	68-4T	U-12	19.2	272	35	237	2.80
Upper shale	68-4T	U-7	18.3	159	24	135	2.74
Lower Shale	68-4T	U-7	16.6	150	21	129	2.75
Upper Shale	68-4T	U-2	21.7	143	29	114	
Lower Shale	68-4T	U-2	16.8	109	25	84	
Upper Shale	68-3T	U-12	15.5	121	26	95	
Lower Shale	68-3T	U-12	16.8	142	26	116	
Lower Shale	68-3T	U-16	15.8	100	23	77	2.80
Upper Shale	68-3T	U-8	14.8	118	25	93	
Lower Shale	68-3T	U-8	14.0	103	23	80	
D. Salts	68-3T	U-1	17.2	121	22	99	2.72
D. Salts	68-3T	U-15	—	116	25	91	2.78
Swell	68-6T	*Comp	—	100	22	78	2.68
D. Shear	69-1T	U-5	16.6	129	25	104	2.75
		Average	15.6	119	25	94	2.74

*U-4, U-5, U-6

Table 14

Fort Peck Spillway
Atterberg Limits and Specific Gravity

Bentonite Samples

Test or Remarks	Test Hole	Sample	Moist	L.L.	P.L.	P.I.	Specific Gravity
D. Shear	68-5T	U-5	25.9	300	43	257	2.90
D. Shear	68-5T	U-4	25.6	363	37	326	2.91
D. Shear	68-4T	U-16	15.9	332	30	302	2.83
D. Shear	68-4T	U-14	23.9	268	37	231	2.86
D. Shear	68-4T	U-12	23.4	278	41	237	2.83
Atterberg	68-4T	U-2	25.8	174	33	141	—
D. Shear	68-3T	U-16	24.2	343	40	303	2.83
D. Shear	68-3T	U-8	26.8	243	36	207	2.92
W.E.S. (1)	71-1T	U-3	27.9	393	43	350	2.88
W.E.S. (2)	71-1T	U-3	27.9	421	45	376	2.88
D. Shear	65-1T	U-20	25.3	252	46	206	2.84
Test Trench	40+70	U-1		177	39	138	
Test Trench	40+70	U-7	52.8	255	42	213	2.88
Test Trench	40+70	U-8		175	43	132	2.82
Test Trench	40+70	U-9	48.0	226	40	186	2.97
		Average	28.7	280	40	240	2.87

(1) Air Dried

(2) Blenderized

3.10.15 Swell Tests: Swell tests were run on the Bear Paw Shale to determine swell characteristics of shale and bentonites and to study the numerous factors which have an effect on swell. The results of the conventional swell test on shale at natural moisture content, and exposed to distilled water, indicated that, in 500 hours, the shale would swell just under 5% for a normal load of 0.1 ton per sq. ft. The other values adjusted to 500 hours plot in a fairly straight line and indicate a load of approximately 20 tons per square foot would be necessary to keep the shale from swelling. The following tabulation gives the percent of consolidation for each load for two samples.

Table 15

Consolidation Pressure T.S.F.	Percent Consolidation Sample D-1	Percent Consolidation Sample D-8
20	0.14	0.30
50	1.04	1.20
100	2.00	2.14
200	3.43	3.97
300	5.44	3.79
400	—	7.62
500	9.00	9.24

CHAPTER 4. - EXCAVATION PROCEDURES

4.1 EXCAVATION GRADES. As far as can be determined, excavation grades for major structures were to lines and grades shown on contract drawings. Changes to original specified slopes were later made on slopes along the spillway and behind the powerhouse as discussed in Chapter 9.

4.2 UNWATERING PROVISIONS. Ground water was not encountered in sufficient quantities to require unwatering provisions for the embankment, powerhouse or spillway excavations. Water seepage occurred along bedrock joints and fractures during excavation of the outlet portals and required pumping from sumps.

4.3 OVERBURDEN EXCAVATION. The Spillway: For the spillway contract weathered shale, as well as glacial till, was treated as overburden. Glacial till and associated gravel lenses varied between 0 to 30 feet in depth, and in places formed part of landslide masses. The till remained stable on excavated slopes. Slopes excavated on glacial till and weathered shale were one vertical to three horizontal. A total of 5,750,000 cubic yards of overburden was removed from the spillway channel area by Government and contractor work forces. Weathered shale and till material was removed by 1-1/2 yard Lorrain Diesel shovels, 2 yard Lima and Bucyrus - Erie shovels, 9 yard Le Tourneau scrapers, Caterpillar tractors, and Euclid and Caterpillar bottom and end dumps with capacities between 6 and 14 cubic yards. Hauls were accomplished by 3 and 3-1/2 ton White and Diamond "T" trucks. The general method of excavation consisted of: Generally cuts were made in terraces from the top down by the shovels to begin the actual open cut to specified side slopes. Channel excavation limits were gradually roughed in. Material was excavated in 10-foot vertical lifts and side slope cut terraces were connected in units that progressed down the spillway. Slopes were finished to grade by power scrapers drawn by Caterpillar tractors or by tractors with blades.

The Main Embankment: The entire base of the original embankment was covered with a natural deposit of 3 to 30 feet of clay which had low shear strength when saturated. Natural clay was stripped away in order to take advantage of underlying pervious material to lower the saturation line in the embankment. Between May, 1934 and November, 1934, a total of 4,133,350 cubic yards of the clay was removed from the base of the main embankment. In general, the clay was removed by elevator graders that were pulled by diesel tractors. Draglines and shovels made the deeper cuts at the foot of the bluffs. The major items of equipment consisted of nine elevating graders, 75-hp diesel tractors, shovels, draglines, and 1,250 3 and 4 yard dump trucks.

4.4 ROCK EXCAVATION.

4.4.1 Tunnels. Excavation operations proceeded simultaneously in four initial pilot tunnels (Photos 15 and 19) in the Bear Paw Shale by drilling, blasting, mucking, and timbering methods. Advances averaged about 35 feet per day in each tunnel. Pilot tunnels were semicircular, 15 feet wide and 13 feet high. In excavating, a 22 hole round was used. Drilling for the round was with hand-held standard jackhammers using 1-1/2-inch, twisted drill steel from 4 to 10 feet in length. An 8-feet round, charged with 46 pounds of 30 percent DuPont gelatin was standard. Detonation was by means of six interval delay electrical primers. Powder factors averaged 0.95 pounds per cubic yard. After shooting, all loose material on the tunnel side walls was removed by hand and a coat of bitumen was sprayed on the tunnel walls to prevent slaking. As an additional precaution to prevent disintegration of the shale, the air in the tunnels was kept at least 90 percent relative humidity by means of combined air and water jets on 200-foot centers. Excavation by means of specially built coal saws was experimented with in the tunnels, but the saws were not successful. The saws were an integral part of machinery with a total weight of 42,000 pounds and required 135 hp to operate. In operation, excessive vibration would not permit the machinery to stay in position once the cut was started. When the second bite of the cut was started, the machinery had settled and binding of the cut resulted. After much experimentation, the method was abandoned. For tunnel enlargement, a

mucking jumbo was used on a 20-foot gauge track. There was room beneath the jumbo for the operation of two Conway mucking machines. The jumbo contained four movable working platforms that could extend from the front of the jumbo to the tunnel face. The platforms were moved by air driven cylinders.

4.4.1.1 A heavily supported shaft (Photo 8) was sunk 210 feet upstream from each of the pilot tunnel portals. A concrete head block (50 feet wide and 30 feet thick) was poured in each shaft to create a headblock which would act as a buttress to the hillside when enlargement of the pilot tunnels began. Enlargement of the pilots to the full bore, 32 feet, 2 inches in diameter with an 8-inch allowance for overbreak, commenced in July 1935. The Contractor planned and started the enlargement as a full face operation. Hand-held jackhammers (Photo 9) that used 1-1/2-inch auger drill steel were used to drill blast holes. The holes were drilled perpendicular to the face and parallel with the axis of the tunnel. An average round consisted of 44 holes, 28 of which were "rib" holes spaced around the circumference of the tunnel. The remaining 16 holes were "relievers" spaced between the periphery and the pilot tunnel opening in the center. Comparatively light charges of 30 percent gelatin powder were used exclusively. Photo 11 shows an example of a tunnel face after blasting. In average ground, the advance per round of drilling and blasting was 5 feet, which was the standard spacing of supporting ring beams. The circular steel ring beams, which were necessary to hold the shale until the concrete lining could be placed, were placed on 5-foot or 3-1/2-foot centers depending on the character of the shale. Sizes of I-beams were provided to meet varying conditions: 10-inch at 29 pounds, 8-inch at 15 pounds and 7-inch at 12.5 pounds. The ring beams were spaced by 24 purlins distributed around the circumference and the rings were blocked and wedged securely against the rock. Strips of 18-gauge sheet metal were placed behind the purlins in the upper half of the tunnel to prevent spalls from falling on workers below.

4.4.1.2 The Contractor, desiring to speed up progress, tried the top heading method of excavation for several months, but gave it up when he found the small increase in speed did not warrant the increased labor cost. It was

later decided to reduce the diameter of the tunnels to 24 feet 8 inches and, in lieu of the 1-inch steel lining, to provide heavy reinforced concrete, except in tunnel No. 1 downstream of the shaft, where a 1 to 1-1/4-inch steel lining would be installed for use as a penstock for future power development.

4.4.1.3 Negotiations were started with the Mason and Walsh Company in the latter part of 1935 with the idea of issuing change orders to take care of changes of plans related to the tunnel lining. However, no basis of mutual understanding could be reached and the Government agreed to pay the Mason and Walsh Company for the work that had been done up to and including the 15th of January, 1936, and to buy from the Contractor all his plant, equipment, and supplies. Accordingly, on the 15th of January, control passed from the hands of the Contractor to those of the Government. The Government took over the entire working force of the Contractor as well. The Government adopted the top heading method tried for awhile by the Contractor. The general plan of this operation was to advance excavation of material above the pilot tunnel a distance of approximately 100 feet. As the top heading progressed, the pilot tunnel roof was floored with loose planking, leaving openings so that the heading muck could be scalped, by means of a tugger-operated scraper, into muck cars spotted below. The top heading was supported by quarter sections of the ring beams resting on 24-inch CB 100-pound needle beams. The needle beams were blocked up so as to maintain a bearing on the floor of the top heading and extended back to completed ring beams where the downstream end was supported on at least three of the completed ring beams. In this way the top section of the ring beam was supported by the needle beam while excavation and placing of the lower sections of the ring beam were carried on. Each needle beam was composed of two 10-foot sections joined with full strength splices so that as operations progressed the downstream section could be removed, moved forward and bolted again at the upstream end.

4.4.1.4 With the elimination of the steel inner lining and the substitution of reinforced concrete, the ring beam supports and purlin connections were welded to provide additional reinforcement.

4.4.1.5 Excavation of Emergency Shafts and Gates. Sinking of the pilot shafts for the emergency gate shafts was started in November, 1934. Head frames, about 25 feet high, were erected over each of the shafts. They were equipped with Ingersoll-Rand tugger hoists for removing material as excavation proceeded. The pilot shafts were 8 feet by 8 feet and timbered at 5-foot intervals by means of 6-inch by 8-inch collars with 2-inch tight sheeting on the inside. After the pilot shafts were holed out into the pilot tunnel, enlargement was accomplished by the "glory hole" methods. As blasting was not permitted, the shale was broken up with pneumatic pavement breakers and shoveled down the pilot shafts where hoppers loaded the waste material into rail cars that carried it out to the intake portals.

4.4.1.6 Excavation of Main Control Shafts and Gates. In October, 1934, the pilot shafts for the main control shafts were started. Head frames about 25 feet high equipped with Ingersoll-Rand tugger hoists were erected at the center location of each shaft and excavation started. The pilot shafts were excavated with pavement breakers and the material hoisted to the top in 4 cubic yard buckets. The shafts were 8 feet by 8 feet and timbered at 5-foot intervals with 6-inch by 8-inch collars with 2-inch tight sheeting on the inside. The sheeting was used to facilitate the passage of muck down the shaft during enlargement operations. The control shafts were designed with a base sufficiently large to avoid excessive loads on the shale foundation. Shaft foundations were approximately 145 feet long, 36 feet wide and 60 feet high. Due to the faulted and blocky condition of the shale through which the shafts were sunk, it was not regarded as safe to fully open up the main shaft excavation to grade and to proceed from there to enlargement of the shaft foundation. The plan decided on was essentially as follows: at a short distance upstream and downstream of the main control shaft, a raise was sloped up from the roof of the pilot tunnel to the top grade of the shaft foundation. This raise was widened out on either side to cover the width of the foundation excavation. Drifts were driven upstream and downstream from these enlargements and small shafts were sunk in which permanent steel columns and cross beams were set to furnish support for the steel roof. After this was accomplished the center core of the foundation was safely

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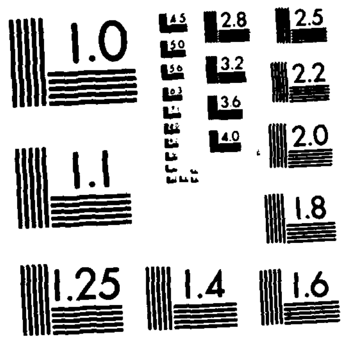

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excavated by ordinary methods. A 1-inch steel lining, which eventually joined the steel lining at the emergency shafts, was then placed throughout the entire shaft foundation and grouted in place.

4.4.1.7 When the pilot shafts had been opened into the tunnels, enlargement of the shafts was started immediately. Four guy derricks were erected, each with a 115-foot mast and a 100-foot boom with a 20 ton capacity at a radius of 45 feet. The shale was broken up with pavement breakers and pushed into the pilot shafts by small bull-dozers where it fell into hoppers that loaded rail cars. The muck was then carried out by way of the inlet portals where it was wasted.

4.4.1.8 A system of circular trusses and vertical walers supporting a light steel lining were placed as excavation progressed to support the shale. Holes in the lining permitted the annular area between the lining and shale to be grouted at the end of every 8-foot advance. The grout, composed of one part cement to 1-1/2 parts sand, was mixed in a 3/4-yard mixer at the surface, placed in a grout machine and the mix forced into place through grout hoses. The grout was introduced through the lowest holes in the lining and built up to thoroughly fill the area. The annular area was entirely filled when grout vented from the top of 6-foot standpipes attached to the top holes of the lining.

4.4.2 Outlet Portals. Due to the nature of the material in the hill (Photo 15), it was deemed inadvisable to excavate continuous blocks. The alternate block plan of excavation was adopted; that is, two 20-foot blocks on either side of a 40-foot block were excavated. After pouring these blocks, the center block could be safely excavated. In this manner, only a portion of the ground was open and unsupported by earth or concrete at any one time. The ground in the weathered zone was excavated by chipping and clamming. As firmer shale was struck, it was removed by line drilling the walls and benches followed by blasting the central portion. Muck was removed by clamshell or skip with the larger blocks of shale chipped to facilitate handling. Excavation was carried on for one set of supports at a time. After pouring

these blocks, the side rakers, outside the block line, were salvaged and work done to remove the shale in the 40-foot block between. The back face of this block was supported by steel wales spliced to the projecting ends of the back walls in the 20-foot blocks, with these wales braced to poured niches in the completed concrete. All shale surfaces were sealed with a sealing solution that was applied by a spray at 40 lbs. pressure within 3 hours after excavation. All sheeting consisted of open 3" x 12" lagging.

4.4.2.1 The manner of excavation adopted for the riverward retaining wall required no structural steel. The shale and weathered material behind the wall was removed by shovel to a stable slope. Then excavation for the base of the blocks was made. Forms were then built, reinforcing steel placed, and concrete poured. The wall was then backfilled.

4.4.2.2 Channel slab excavation was accomplished by shovel and clamshell with chipping where necessary. No support was needed for this shallow excavation. The excavation was taken to firm shale. In many cases, it was necessary to remove bentonite seams. Fault gauge was removed from 1 to 3 feet along the fault plane below the base of the slab.

4.4.2.3 Cutoff wall excavation from the base of the floor slab at elevation 2023.83 to a maximum depth at elevation 1990 was accomplished entirely by chipping and clamming. Alternate blocks were sunk with sets of structural steel supports at 4-foot centers except in two blocks where the ground was bad due to faulting; here, steel was placed at 2-foot centers. Four-inch wire mesh sheeting to catch any sloughing material was placed between the steel sets. After concrete was poured in the alternate blocks, the block between was excavated and poured. Seepage water in quantities requiring pumping from sumps was found in several of the blocks, especially those in which the ground was faulted. Three major faults were exposed, but no ground movement occurred beyond minor sloughing behind the sheeting. During all excavation, careful geological notes were recorded. All faults and bentonites that were exposed were traced and mapped.

4.4.3 Inlet Portals. Earth was removed for the inlet portal pit by power shovel which left one-on-one slopes on the edge of the excavation. Slopes above the elevation of the top of the wall were cut to 1V on 3H. No slides occurred while excavating for the inlet portal pit. The characteristics of the shale for this excavation were practically the same as in the outlet portal.

Before work began on the excavation for the inlet portal retaining wall, the following considerations were adopted:

4.4.3.1 Surface slides had occurred near the tunnel portals (Photos 3, 4, 5, 17) and the general plan of excavation was carried on in such a manner as to have the minimum of ground open and unsupported and yet have that portion of work completed so as to not impede tunnel progress. One block adjoining each tunnel portal was excavated and poured. The operation was repeated on blocks on the opposite side of the portal. The material excavated well and no major difficulties were encountered. One small slide of weathered material in one of the intermediate blocks bent the two upper back wales, necessitating the straightening of these members.

4.4.4 Main Tunnel Enlargement. Prior to enlarging the pilot tunnel to the final size of the main bore, the geological information (Plate 36) obtained in pilot tunnels was carefully considered. Photo 6 illustrates a fault surface encountered in the Bear Paw shale (during investigation of the 1938 abutment slide) and is representative of conditions that could be encountered during tunneling. It was determined that a heavier type of bracing was necessary than that initially specified. Heavier bracing was to be placed only where necessary. The ground encountered in the pilot tunnels showed that bracing was needed throughout. Purlins between the beams put the heavy rings at 3-12 foot centers. Light and intermediate weight rings were to be at 5 foot centers in better ground. Subsequent enlargement of the main tunnel showed that the design location of the various types of beams could not be adhered to, and the spacing and type of ring placed was dictated by

field conditions. The tunnels were lined with reinforced concrete. For protection in the area as yet unconcreted, steel sheeting was spaced 4 inches apart, drawn tight, and clipped at the spring line purlin. The sheeting was placed over the purlins between the ring beams.

4.4.4.1 Work on the main tunnel enlargement began at the lower portals upon completion of the cut and cover tunnels. The sequence of operations was as follows. Excavation was performed by drilling blast holes and blasting. Experiments with radially drilled holes were not successful in the shale, due to the blocky, jointed structure. The back was trimmed and temporarily supported by crowbars cantilevered from the last ring beam in place. The blasted shale was removed by Conway Mucking Machines, loaded onto a conveyor, and dumped into rail cars. As the muck was removed, the upper arch and sides and invert of the tunnel were trimmed to permit placing of steel and supported wherever necessary by crowbars. When mucking was finished, the shale surface was sealed with bitumen, the ring beams hoisted into place, and bolted to purlins from the preceding ring. The texture of the shale before treatment is shown by Photo 7. In cases where the rock was badly broken, protective grouting was placed before the concreting. Tight sheeting was placed at the outer flange of the ring beam and grout was pumped in. The ends were then dammed until the annulus between the sheeting and the shale was filled with grout. The grout strengthened the bracing and held the disintegrated shale together and supplementing the blocking until the concrete operations reached the grouted area. The concrete lining was later effectively pressure grouted.

4.4.4.2 Closely allied with the type-of-ground and geological structure was the general method of tunneling. A choice had to be made between the top heading and bench method, and the full heading method. Both were tried. The former was adopted and used in the greater length of tunnel. The top heading was carried about 100 feet ahead of the bench, with two segments of each ring placed in the upper heading. Then the remaining three segments were placed in the bottom heading. In this manner, the top heading was approximately 12 feet high; the bottom heading was 20 feet from top of bench to invert. The

advantages of the top heading and bench method over the full heading method are briefly listed:

(1) Safety: A small face was exposed in each heading and had less tendency to cave and slough in bad ground. Workers could see the whole face from any position. This was not the case in the full heading. Workers could also detect any ground movement. The heights of falls for workers was less and resulted in a decrease in serious injuries. Accidents showed a noticeable decrease after changing to the top heading and bench method.

(2) Decreased overbreak: Blasting was lighter, less ground was opened up at one time, and fewer and less serious cave-ins occurred. Overbreak averaged about 6 inches.

(3) Speedier advance: Support was accomplished more quickly and fewer temporary timbers were needed in the sidewalls. In the full heading no work could be done on the sides or bottom until the upper arch and back were caught up. In the top heading and bench method, all excess time needed to support the back was done in the top heading; after blasting in the bottom, mucking could begin at once.

4.4.5 Excavation of Spillway. Approximately 5,000,000 cubic yards of firm shale were removed from the spillway (Photo 24). Slopes in firm shale that were less than 50 feet high were graded at 1 vertical on 1 horizontal. Slopes between 50 to 70 feet high were graded to 1 vertical on 1.5 horizontal. Slopes greater than 70 feet high were cut to 1 vertical on 2 horizontal. Rock was excavated by drilling and blasting. The type of explosive used was 30 percent gelatine dynamite in 0.45 lb. sticks. Blast holes were drilled between 3 and 9-foot depths and loaded with 0.9 to 4.5 lbs. of dynamite per hole. On the average, approximately 0.18 lb. of dynamite was used per cubic yard of excavated shale. The contractor used five 2-cubic yard Bucyrus-Erie and four 2-cubic yard diesel powered shovels which were occasionally supplemented by four 1-1/2 cubic yard shovels and drag lines.

For the lined channel, excavated material was hauled approximately 1 mile northwest of the spillway and dumped in the SW¼ of the SW¼ of Section 6, T 26 N, R 42 E. For the unlined portion of the spillway, spoil was placed in an area known as Spoil Area "C" which is east of the centerline of the lower spillway area along the Missouri River. Spoil Area "C" is located in the W½ of the NW¼ of Section 5, R 42 E, T 26 N. Excavation hauls for both portions of the spillway were accomplished by Euclid and Caterpillar bottom and end dumps with capacities that varied between 7 and 14 yards.

4.5 FOUNDATION PREPARATION.

4.5.1 Spillway-Lined Channel: One to three feet of cover was left on slopes that were graded during the winter months. The slopes were finished the following spring with draglines and power tools such as Sullivan coal cutting saws and Lima pull shovels.

In order to prevent the exposed shale surfaces from disintegrating before concrete was placed, a sealing solution, "Presatite A" - a gilsonite solution - was applied to the shale surface with one to three applications per surface, depending upon the length of time the surface would be exposed. All surfaces were blown clean with air prior to applying the seal. The sealing solution was applied as soon as possible after the shale surface was exposed and before flaking ("checking") of the surface occurred. Re-chipping of the surface was required when flaking ("checking") had occurred due to over-exposure. The surface seal was applied by spray nozzles which applied a finely divided spray. Where the shale was protected from the direct rays of the sun and rain showers (as in deep trenches), one coat of surface seal was found to give excellent protection against disintegration. At locations where the shale was more exposed, two coats of seal provided sufficient protection for 7 to 14 days if not exposed to rain. For shale surfaces with extreme exposure, such as side wall slopes, three coats of sealant were given since construction procedures required the side walls to remain exposed nearly 14 days. When three coats of sealant were required, the third coat was not applied until the first two coats showed signs of failure. The

surface was then blown clean and the third coat applied. Rain was very detrimental and caused the seal to peel. Exposed surfaces were covered with tarpaulins as soon as possible when there was a threat of rain.

Since work at the lower end of the spillway channel involved high walls and the upper portions of the walls would be exposed beyond the effective period of the sealing solution, gunite with wire mesh was applied over the sealing solution. This method effectively sealed surfaces that were exposed the longest. Certain monoliths near the lower end of the spillway channel floor that were excavated in the spring of 1937 were damp and it was difficult to maintain a good seal. To protect these surfaces, a thin layer of pea gravel concrete was spread over the monolith as soon as a single coat of sealing solution had dried. This method of protection was so successful that it was adopted as standard practice for the completion of the work on all horizontal shale surfaces.

4.5.2 Tunnels. Protection of the shale surface was accomplished by sealing the rock and controlling humidity and temperature. Tests with various types of sealing solution and gunite were made at intervals during the pilot tunnel excavation. In all cases the Hunt Process Solution was found the most satisfactory. Gunite tests were unsuccessful. The shale was sealed as soon after exposure as possible (Photos 7, 9, 10, and 18 show freshly exposed shale) - generally in less than 3 hours. Second and third coats were applied later at intervals of not more than 10 hours between coats. Application of sealing solution by spray (at 40 pounds pressure) to a freshly scaled surface gave good results. To further protect the shale and prevent any slaking, relative humidity of the tunnel air was kept at 90 percent or more by means of atomizers located at intervals along the tunnel. An atomized spray of water was forced into the air to maintain the desired relative humidity. Air in the tunnel was kept at a constant temperature. Average temperatures within the tunnel were between 50 degrees and 55 degrees. Doors were placed at the portals and at intervals along the tunnel to prevent a rapid circulation of tunnel air which would lower the humidity. In the winter months, the air forced into the tunnel was heated to prevent freezing of water lines and atomizers in the vicinity of the tunnel mouth.

4.5.3 Main Embankment. The original site of the main embankment was covered by dense growths of cottonwood trees, willows, and underbrush, and it was necessary to clear the base of the dam of all vegetation. Stumps were removed by blasting or tractors. Tractors were capable of removing stumps with roots as long as 21 feet. Topsoil was stripped from the site and a clay layer, 3 to 30 feet thick was removed down to more pervious material.

4.6 SAFETY PRECAUTIONS. A safety office was located on site and was staffed by a safety inspector and five assistants. However, it was not possible to obtain specific information on safety precautions for the construction of various phases of the project. It was reported that 65 fatalities occurred during construction.

4.7 PERMANENT FOUNDATION ANCHORS.

4.7.1 Spillway Gate Structure. A total of 467 concrete piles, each 5 feet in diameter and from 30 to 40 feet deep, provide stability and sliding resistance for the structure. The reinforced concrete piles are spaced on 18.33-foot centers longitudinally and on 13-foot centers transversely with intermediate piles located at the center of every group of four. The piles extend 2 feet into the base slab and are an integral portion of the substructure. Pile holes were drilled from 30 to 40 feet in depth. The method of drilling the pile holes is given in "Fort Peck Spillway" by John A. Hardin, Captain, Corps of Engineers, as published in The Military Engineer, January-February, 1937. It was not possible to locate this article. For the upper 10 feet of the pile hole, a sealing solution was applied to the exposed shale to prevent the shale surface from drying out prior to placing concrete in the holes. Sealing solution was not applied to the lower portion of drill holes for economy reasons and in order to expedite the work. Concrete was poured into the pile holes within two days of drilling the hole. Dumping of concrete was alternated between two piles to allow sufficient time for thorough vibrations of the concrete without interrupting concrete pours. Six to eight piles were poured from one setup to match the pile drilling rate of 120 feet per drilling rig. Reinforcing steel for the piles was fabricated in

advance. The entire steel cage (circular) was welded together and lowered by crane into the hole. A minimum clearance of 3 inches was allowed between the outside of the reinforcing steel and the shale walls of the hole.

CHAPTER 5. - SPECIAL FOUNDATION CONSTRUCTION

5.1 MAIN EMBANKMENT CUTOFF WALL. The first construction work done on the dam was the construction of a sheet piling cutoff wall (See Plate 3 and Photos 1A and 1B). Subsurface explorations across the river valley disclosed that numerous interconnecting layers of coarse sand and gravel existed above the firm shale bedrock; consequently, measures to retard percolation and to seal all possible subterranean channels were considered. The Board of Consultants, at a meeting on January 11, 1934, recommended that a steel-sheet-piling diaphragm be driven to shale throughout the length of the main dam. The wall is located parallel to, and 37.5 feet upstream of, the axis of the dam. The wall extends across the flood plain between the right and left abutments. On the right abutment, the wall follows a natural ridge up the abutment. On the left abutment, the wall continues up the abutment to a bench and then westward to approximately Station 103+00. The wall consists of interlocking steel sheet piling that was driven with air hammers in combination with high pressure water jets (photos 1B and 1C). Piles in the river channel were cut off by a marine diver to levels varying from elevation 2022 to elevation 2040. These elevations roughly follow the contour of the river bed.

5.1.1 On May 23, 1935, a section of the cutoff wall on the left bank collapsed due to a comparatively small head differential between areas upstream and downstream of the cutoff wall. The wall was righted, and it became policy to cut equalizer holes through the wall at about 50-foot intervals for this area of work. The cutoff wall was extended in height by welding extensions on the existing wall. Work was carried on as impervious fill was placed to the top of the pilings. Since the lower portions of the wall in this area were driven into impervious clays and the upper portion of the wall was located in impervious fill, the equilizer holes were not closed.

5.1.2 To obtain the maximum efficiency in reducing percolation, and to keep the line of saturation at a maximum distance from the downstream surface of the dam, the cutoff wall was located 37-1/2 feet upstream from the dam axis. As an added precaution, the top of the wall extends a minimum of 20 feet into the dam above the alluvium plane to prevent excessive seepage along the junction between the hydraulic fill and the natural alluvium. Four range walls, aligned almost parallel to one another and having their east ends situated about 200 feet out on the flood plain, were constructed up the left abutment in the four coulees located between the axis and the downstream [illegible]e of the dam.

5.1.3 The cut-off wall extends from station 2+04 to station 103+50 a distance of 10,146 feet. The length of the four range walls totals 2,[illegible] feet. Two different weights of shallow arch piling were used. The bottom tier of alternating 70- and 80-foot lengths weighed 23 pounds per square foot, and the top tiers weighed 28 pounds. The webs of these sections are 3/8 and 1/2 inch thick, respectively. The heavier 1/2-inch piling was used for the upper tier or tiers to better withstand the severe punishment to which the piling would be subjected while driving the lower tier to great depths. Piling was driven to a maximum depth of 163 feet. A total of 1,374,079 square feet of piling was placed.

5.1.4 It was not feasible to begin building the cutoff wall until stripping of the dam site was out of the way, and it was equally important that construction of the cutoff wall be completed in such a manner as to offer a minimum of interference with placement of the fill. In order to sandwich the construction of the wall between these two major operations, the construction of the wall under the original specifications was divided into six priorities, each with a stipulated date of completion.

Priority No. 1, was situated between station 73+00 and station 85+00 and was scheduled to be completed November 11, 1934. Construction of the wall was timed to fall behind the stripping in that vicinity and to precede any extensive fill operations. Priority No. 2, was situated between station 73+00 and station 64+50 and scheduled to be completed by December 31, 1934.

Priority 1 and 2 were worked simultaneously. Priorities Nos. 3 and 4, located between station 56+00 and station 17+00, were scheduled for completion by March 27, 1935 in order that the flood plain east of the river would be ready for the 1935 dredging season. Priority No. 5 included all of the wall above the flood plain and was divided into subpriorities for convenience of administration. A supplemental agreement between the Government and the contractor provided for driving the 811 lineal feet of wall necessary to complete the river section between priorities Nos. 2 and 3. This addition was designated Priority No. 6, with October 19, 1935 as its completion date.

5.1.5 In general, the method of installation was to set the bottom tier of sheeting by the use of cranes, either railroad or tractor (tractors were better). Jets were used to make a hole or slot into which the sheets of piling were dropped. For placing the top tiers of sheeting, large steel frame gantries were employed (Photo 1A).

5.1.6 Jetting of the holes at the start of operations was accomplished with round jets made of extra heavy pipe with an outside diameter of 4.5 inches and an inside diameter of 3.16 inches. Pipe lengths varied from 86 feet to 172 feet. Shorter length jets were handled with locomotive and caterpillar cranes with boom lengths of approximately 100 feet. The longer jets were handled from gantries. Nozzle apertures bar jets were 1-5/8 inches in diameter and required 600 gallons of water per minute. Later, diameters of 1-1/2 inches, 1-3/8 inches and 1-1/4 inches were tried, with the result that the smaller size eventually came into general use. This was due to the fact that the contractor's pumping plant was seldom able to supply the water required rather than any superiority of the 1-1/4-inch nozzle.

5.1.7 The later development of the hydraulic spade (Photo 1C) contributed much to the satisfactory driving of the piling. A regular 4-inch jet pipe was welded in the concave face of a 16-inch steel sheet pile. The jet delivered water into a manifold that was made of 3/8-inch cover plate fastened at the bottom of the pile. Five wedge-shaped teeth projected 5

inches beyond the end of the manifold and pile. The manifold contained seven 1/2 inch orifices for the discharge of water. Four of the orifices emerged between the five teeth, two into the interlocks, and one at the top of the manifold to reduce friction of the jet. All but the lower few feet of the interlock was removed from the pile used for a jet to reduce friction between the jetting pile and the pile in place.

5.1.8 To set the first tier of piling, using the original round jet, the jet was lowered between two very heavy timber walers spaced not more than 6 inches apart. Three holes were jetted successively at distances of 21, 13, and 5 inches in advance of the last sheet set. Care in keeping the jet in a true vertical position was appreciated more as the work progressed, because it prevented twisting and leaning of piles. Immediately after the three holes (under favorable conditions, two holes were sufficient) were jetted, a sheet was threaded and dropped into position in the jetted hole. Very little driving was required in placing the alternating 70 and 80 foot first tier of piling.

5.1.9 As the work progressed, the amount of equipment necessary to accomplish the work increased. At the start of operations, water for jetting purposes was supplied by two Dayton-Dowd two-stage centrifugal pumps of 600 gallons-per-minute capacity each and powered by a 150-hp. Twin City Diesel engine. The plant also contained two Cameron two-stage centrifugal pumps of 600 gallons-per-minute capacity each, powered by a 100-hp. Louis Allis induction motor. The pumps supplied water at 175 to 200 pounds per square inch pressure. One compressor supplied air at approximately 100 pounds/in2 pressure. The compressor was an Ingersoll-Rand Imperial having a capacity of 1,230 cubic feet per minute and was powered with a General Electric, 220-hp. synchronous motor. The two locomotive cranes, two caterpillar cranes, and one still gantry were supplemented with four more caterpillar cranes and two more gantries.

5.1.10 The placement of the bottom ti · of piling was followed up with threading, welding, and driving of the second and third tiers. The employment of steel frame gantries for this work was probably the greatest single factor contributing to the successful completion of the work. The gantries made it possible to thread exceptionally long second-tier lengths of piling and handle water jets up to 170 feet in length. They also afforded unusual steadiness which was essential in efficient prosecution of the work. The hardest driving was usually done with a 10 B-3 hammer if available.

5.1.11 Welding was the principal method employed in splicing the piles. It was specified that: (1) The piles be driven in lengths such as to require no more than two splices for each full length of pile placed in the wall. (2) The lengths be selected and placed in such a manner that no two adjacent splices, in adjoining piles, would be within 10 feet vertically of each other. (3) Each splice have a tensile strength equal to that of the pile section.

5.1.12 Each of these requirements was modified later to some extent. Eventually permission was granted for the use of three splices, but only in a relatively few instances were more than three sections of piling used to form one full length pile. Near the middle of the job, the contractor was permitted to change the staggering of splices from 10 feet in adjacent piles to 10 feet in adjacent pairs of piles.

5.1.13 The original contract did not provide for construction of the cutoff wall across the river channel due to insufficient data available at the time the specifications were issued. However, a supplemental agreement with the contractor, dated May 8, 1935, provided that the 811-foot river gap (station 64+11 to station 56+00) be constructed. The agreement called for 131,400 square feet of sheet steel piling.

5.1.13.1 The procedure followed in setting piling across the river was usually to thread from two to eight piles ahead of driving operations. In order that the river channel would be clear, the top of the piling between station 57+00 and station 58+90 was limited to elevation 2024 or approximately 13 feet below the surface of the river. Air hammers, with exhaust extension to the water surface, drove pairs of piling to their final grade. It was impossible to estimate beforehand the exact length of pile to be used in all cases; it was therefore necessary to burn off the tops of 79 of the 142 piles in that area.

5.1.13.2 A trench was excavated on each side of the wall to provide a space into which the fines from dredged material would be deposited along the wall. Draglines were used to do this work. The trench on each side of the wall was 20 feet wide at the ground surface and continued down with a side slope of 1 on 1 until the water table was reached.

5.2 PLANS FOR DIVERSION. During the first phase of embankment construction at Fort Peck, a free channel was maintained for the river flow while fill was placed on both sides of the channel. During the closure phase, the flow of the river was diverted entirely through tunnels as the fill was placed in the area of the channel. After completion of the main embankment, river flow was entirely through tunnels. A total of four diversion tunnels, an intake structure, main gate shafts, emergency gate shafts, and an outlet structure were constructed for diversion and later used for power generation. The four tunnels are constructed in shale under the right abutment of the dam. With the exception of tunnel No. 1, the tunnels are lined with reinforced concrete. Tunnel No. 1 is provided with a steel plate lining from the control shaft to the outlet. Typical tunnel cross sections are shown by plate 202 and 203. Diversion tunnel construction was started under contract with the Mason & Walsh Company on January 15, 1936, and completed with Government plant and hired labor in June 1937. At this time the earth plug at the tunnel intake was blasted open and the river flowed through the tunnels. Closure on the embankment's upstream toe was affected on June 26, 1937.

CHAPTER 6. - CHARACTER OF FOUNDATION

6.1 MAIN EMBANKMENT. One of the first steps prior to actual construction of the dam was to clear the site and adjacent upstream and downstream borrow pit areas of heavy growths of cottonwoods and willows. Clearing operations began October 23, 1933. Overburden consisting of blue and brown clay ("gumbo") and alluvial silt was stripped by shovels, drag lines and elevating graders. Clays and silts were removed down to clean sand. Approximately 4,100,000 yards of material were removed. Next, the portion of the dam between stations 72+00 and 84+00 on the left bank was scarified. The foundation of the dike section consists of hard glacial till. The till was scarified prior to placing fill material. Foundation materials for the main embankment are shown by Plates 3 through 9. With the exception of the main embankment, all final foundation surfaces at Fort Peck are founded in the Bear Paw Shale.

6.2 Spillway.

6.2.1 Lined Channel. Excavation for the channel began during the winter months and an eight foot protective cover of shale was left above final grades. Difficulties developed when the cover was removed. The shale cover could not be removed with power shovels within 3 feet of final grade without loosening blocky shale at final grade which would result in disintegration of the shale. The solution was to strip horizontal surfaces to one foot of final grade and slope the walls to within 3 feet of final grade, and then use power tools to excavate to grade. The principal power tools were coal cutting saws and Linia pull shovels. Both tools were able to produce neat-line excavations without overbreaks. In addition, air spades were used to cut to final grade. From the standpoint of finish, accuracy and number of blocky fallouts, coal saw cuts were superior. Profiles of the spillway and the geology of the spillway are shown on Plates 85 through 89 and 92 through 101.

6.2.1.1 During the final excavation of the west (left) wall monoliths, shale bedding planes occurred in such a manner as to release blocks of shale which weighed up to hundreds of pounds and caused many accidents to workmen. Photos 21 through 25 illustrate slide problems that were encountered in the spillway area. Photo 22 shows the blocky nature of the shale. Exposed shale surfaces were cleaned by air and sealed as discussed under Section 4.5, "Foundation Preparation." In badly faulted areas, a 1 foot cover was removed by hand labor.

6.2.2 GATE STRUCTURE. In general, the shale at the gate structure was the same as found for the lined channel section, although somewhat softer. However, it was classified as firm shale. Preliminary excavation had revealed the presence of numerous faults in the vicinity of the gate structure. A total of twelve fault planes cut the crest line of the spillway. Plates 85, 87, 88, 92, and 99 show fault locations. Faults had displacements from a few feet to approximately 60 feet and clay gauge zones of less than an inch to several feet. Fault planes were tight and seepage was not a problem. The shale was blocky and separated along joints in a blocky structure, which made it difficult to control excavation to required elevation. The presence of clay gauge in faulted zones caused "raveling" when excavating to grade, and it was practically impossible to secure a good seal with a sealing solution. Although the photo was not taken in the spillway area, Photo 6 serves to illustrate the nature of clay gouge in faulted areas. At times emergency timber bracing was used until regular bracing could be installed. Failure did occur at times as soon as the saw cut was made.

Bedding planes dipped generally to the southeast, and as a result, fallouts and overbreaks from structural weakness were greater on the west side of the channel.

6.2.3 Gate Structure Cutoff. The cutoff wall at the gate structure is a concrete wall 10 feet wide by 30 feet deep, and extends into the shale under the upstream edge of the pier slabs. This wall extends entirely across the channel end under the abutments and is projected up the slope to approximately elevation 2260 in the form of a 4' x 12' reinforced concrete collar.

The primary purpose of this wall is to prevent percolation under the gate structure and abutments. After the difficulties experienced at the gate structure with trench excavation in the vicinity of faults, it was deemed advisable to make additional core borings at the cutoff structure since it was known that one major and one smaller fault existed in the area to be excavated. The purpose of the core borings was to determine the exact nature of the material in which trenches 130 feet deep were to be excavated and whether or not water was present in the major fault. Inspection of the cores revealed that several bentonite beds existed and that water was present in the fault plane. It was also ascertained that in certain areas the shale was very blocky with slickensided joints and had to be braced.

6.2.3.1 The shale at the cutoff structure was much better than that encountered at the gate structure, and it was possible to excavate and hold the trenches open for a depth of 130 feet. Under the restricted plan of operation, very little trouble was experienced in holding the shale, and only when this plan was liberalized to provide the contractor more working space, did any failure occur in the steel bracing.

6.2.3.2 Since the borings made in the immediate vicinity of the spillway were primarily for the purpose of determining the depth of firm shale below the surface, little information was obtained regarding the structure of this material. Only a few holes in the gate structure area were drilled to greater foundation depths, and the resulting data was insufficient. The gate structure was so designed that the presence of faults had no effect on the completed work, but certain abutment structures had to be re-designed during the course of the contract. This was due to incomplete knowledge of the foundation conditions. The lack of knowledge concerning the faulted condition of the shale was, as it later developed, more serious in the case of the cutoff structure in that the construction procedure had to be severely restricted in order to insure the safety of the work. Because of the haste in getting the work started, foundation studies were inadequate, and difficulties were later encountered.

6.2.3.3 The nature of the shale in which excavation took place necessitated extreme care in excavation and bracing to safeguard the workmen and to insure that the sides would hold until concrete could be poured. The first operation in excavation was to make two saw cuts 5 feet deep outlining the trench position. The material between these cuts was then removed to a depth that would permit the first tier of steel to be installed. Excavation provided a 4-inch clearance between the shale and the steel on each side. Steel was held in place by hanger rods and by blocking and wedging against the shale face. The remaining material was then removed and another saw cut made. The faces were sealed as excavation progressed and as soon as the second tier of steel was placed corrugated metal sheets were installed. The bottoms were caulked to prevent leakage, and a grout mix of two parts sand to one cement was poured behind the sheets. The same method was followed for each of the several tiers until the trench was excavated to the depth required. Blasting was permitted to loosen the material to be excavated between the saw cuts, but care was exercised to keep holes shallower than the cut to avoid transmitting the shock to the walls of the trench.

6.2.3.4 The above method of sheeting and grouting left a solid impervious mass to pour concrete against. Initial movement of the shale was prevented and reduced the possibility of percolation along the wall.

6.2.4 Abutment Walls - Trench Type: That section of the spillway gate structure generally referred to as the trench type abutment walls is the cellular structure immediately downstream from, and adjoining, the main gate structure abutments. The front walls of this structure form the channel sidewalls from the abutment piers to a point opposite the downstream ends of the training walls. It was known that the shale in this area was seriously faulted, but grouting or filling voids with concrete behind the sheets was omitted and the omission of grout or concrete contributed to movement which occurred during construction and threatened trenching operations. The danger of trench failure persisted until concrete was poured. The omission of grout

left overbreak voids behind the sheeting and decreased the bearing surface against the structural steel bracing. Under the method followed, the larger voids were blocked and cribbed with timber, which in many cases did not transmit the load to the steel. Movement started and caused vertical displacement of the steel. Many vertical braces between the tiers of steel and additional struts were required in order to hold the trenches until concrete could be poured.

6.2.4.1 Due to the difficulties encounter-' in trenching the first monolith, the high cost of grouting and bracing, delay to the contractor, and the condition of the ground in which the remaining part of the structure was to be built, a change order was issued authorizing a combination of open cut and trenching methods for the future work.

6.3 TUNNELS AND PORTALS.

6.3.1 Inlet and Outlet Portals: It was necessary to locate portals so that there would be sufficient covering (about 30 feet) over the tunnels at the portals. After the portals were located, the retaining wall location was fixed. Excavation to a 1:1 slope of portal pits was by power shovels.

6.3.1.1 Outlet Portals: Weathered shale, subfirm shale and some firm shale were all removed by shovel (Photo 15). A large percentage of the firm shale was excavated without blasting. It was first considered necessary to blast all firm shale, but excavation showed that the shovel could handle all but a few spots of firm, unjointed shale. Shortly after preliminary pit excavation began, a minor earth slide occurred. The slide occurred along a fault plane between the gumbo surface material and the underlying weathered shale. The volume of earth in the movement was small, (roughly 16 feet wide, 14 feet long and 3 feet deep). At the time, little attention was given to the slide, and excavation continued. Two weeks later, a second slide occurred in close proximity to the area previously covered by the minor slide. The movement was of greater magnitude and showed well-defined slippage planes on three points of the arc. After this slide the slopes were changed to 1V on 3H.

Then, a third slide (the major ground movement) occurred (Photo 14). Excavation of the slopes to 1V on 3H continued, and simultaneously, government core drilling in the lower portal slide area began. Cores were carefully inspected and evidence of extensive faulting was disclosed. Consideration was given to moving the portals and borings were made with this objective. Results of the investigations showed other areas to be as badly faulted as the existing location, so the walls were redesigned and moved away from the hill. Subsequent geological observations in tunnel excavations confirmed the conditions shown from the drill hole data. Fault patterns in the tunnel area are shown on Plate 36. Plates 160 through 170 show the fracture lines and dates of the surface movements, as well as the subsurface trace of faults on slopes landward of the tunnels. Geological notes indicated that, in all slides, the bottom plane of movement was along a bentonite bed with the back of the movement cut by a fault plane. A driving wedge formed by fault planes at the back of the movement caused the mass to move downward along the sliding plane provided by the bentonite bed.

After stripping operations and excavation of the lower portal pit, excavation for the hillward retaining wall was started. Excavation was through material that averaged 4 feet of weathered shale which graded into firm shale that, in turn, showed minor jointing. In faulted areas fractured shale was found. Some water was encountered which seeped into the lower part of those blocks near the outlet end of both retaining walls.

One major slide was encountered in the excavation of one of the 20-foot blocks (Photo 17). The movement occurred when an area behind the block began to develop cracks and the wales started to show signs of taking weight. Supports under the wales were set on loose material over a fault plane and the supports settled. Unauthorized removal of the supports by the contractor permitted the whole mass to settle tc such an extent that the steel was distorted and had to be replaced.

The experience with excavation showed that structural conditions beneath the surface of the shale made it easy for a mass movement of shale to take place. Consequently, the wall was redesigned for increased strength. The ground encountered in inlet wall excavation was not a serious impediment to work. Approximately 15 feet of weathered surface material was removed. The work required the use of 3" x 12" sheeting in the upper part of the excavation to prevent material from sloughing. Excavation below the zone of weathering was carried on in such a manner that no major difficulties were encountered. Below the zone of weathering approximately 20 feet of blocky and jointed shale was exposed. Sections on plates 160 through 164 show the position of bentonite beds and fault planes in the tunnel. Plates 195 to 201 show fault locations in the tunnel.

6.3.2 Pilot Tunnels. After the Inlet Portals were excavated, work began on the Pilot Tunnel. A center bore tunnel was used to prevent damaging the shale at the outer limit of final excavation in the main tunnel. The first ground encountered at the portal was highly faulted, fractured, jointed, and weathered. Firmer rock was drilled and blasted (Photo 11). Blasting in the fractured area precipitated the upper portal slide. Portal excavation in Tunnels No. 3 and No. 4 had just started when ground movement occurred. Beginning at Portal No. 2, the disturbed area extended 170 feet toward portal No. 4, and 60 feet up the 1V on 1H slope above the portals. The slide was preceeded by a blast in No. 3 portal, and the area north and east of the portal was considerably disturbed. Soon afterward, the back began to spall off in small slabs, that indicated the rock was under considerable stress. Spiling work was started. The timbers in Tunnel No. 3 began to show the strain of additional weight, and shortly afterward collapsed; the north wall of tunnel No. 3 was crushed and material from the slope above the tunnel filled the excavation.

6.3.2.1 Work was discontinued in all portals until a wooden retaining wall was built. The wall effectively held the hill in place until the permanent inlet portal retaining wall was constructed. Analysis of the slide showed a fault plane at the back of the movement and movement was taking place along a bentonite bed.

Work then proceeded on the pilot tunnels through crushed and broken ground. As soon as firmer shale was encountered, timber sets were placed at 5 foot centers with 2" x 12" lagging overhead which rested on the arch segments. Distances, center to center, of timber bents varied throughout the tunnels from 10 inches (in the cheek to cheek area at the portals) up to 8 feet. Side lagging, backfilled, was placed where the rock was bad. Lagging was spaced 4 inches to permit applying the second and third coats of sealing solution.

6.3.2.2 Thorough notes on the geological structures in the face, back, and walls were taken by the inspectors after each blast. The sketches made and notes taken were given to the geological department each day, where they were drafted on an accurate section and plan of each tunnel. These sections and plans showed all bentonite beds, faults, major joints, type of ground, geological notes, timber bents, and stationing. The stratigraphic column established from data obtained from the core drill borings, but which could not be correlated previously to this time, was completed during the driving of the pilot tunnels. The series of shale beds, bentonite beds, fossil horizon, and glauconite beds was found to be quite uniform in the exposures of the tunnel area. The distances between the bentonite beds varied only a few tenths of a foot with the bentonite and glauconite deposits being quite uniform in thickness.

6.3.2.3 Before driving the pilot tunnels, some fault locations were suspected, but a far greater number of faults were found than were anticipated (Plates 196 to 201). The average distance between fault zones was 200 feet. The intensity of the faulting was variable, with vertical displacement being as much as 70.5 feet. A part of the section was always exposed in the foot walls and in the hanging wall. All faults were of the normal type (hanging wall moved downward with respect to the foot wall) and exhibited the customary auxiliary faults normal to the main plane of faulting. Fracturing and jointing was more intense on the hanging wall of the fault than in the foot wall. Due to insufficient knowledge as to the

height and exact location of the major faults at the time of driving of the tunnels, 8-foot long rounds were often blasted quite heavily into a faulted area. Because of this, the back and sides were loosened beyond pilot tunnel lines and loose blocks of shale that were cut by fault planes sloughed into the tunnel. Bedrock conditions necessitated spiling ahead and setting cribbing above before the next set of timbers could be placed. The overbreak at times was from 3 feet to 8 feet. Tight, dry faults with little gouge caused very little difficulty unless bentonite seams overhead formed a plane along which the shale might slough. Blocky ground, with gouge up to 3 feet, either with or without moisture, was often very hazardous. In two cases, work in the area of a freshly exposed fault caused sloughing which took out the first standing set of timber and left 16 to 20 feet of unsupported, loosely hanging shale. Dowels were placed firmly in the foot and hanging wall of several major faults. Readings taken on these dowels showed no change, indicating that no movement was taking place underground.

6.3.2.4 Jointing was well developed in the tunnel area. The effect of jointing was detrimental in most cases and was of such a character that the shale, excavated in cube-like blocks, often sloughed beyond tunnel lines that gave undesirable overbreak. Some rounds would break short of the intended face due to jointing that cut across the blast holes. On the other hand, if the major joint plane was beyond the bottom of the hole, the face would slough to this plane and resulted in large pieces of muck. Jointing often dissipated the force of the blast and caused breakage that was too tight for timber placement. Chipping or plugging was then necessary and resulted in lost time.

6.3.2.5 The bentonite beds encountered in the tunnel area varied in thickness from a fraction of an inch to 1.3 feet in thickness (Plates 196 to 198 show the 1.3-foot thick betonite) and all but the thinnest beds formed a plane on which the shale exhibited a tendency to slough. In general, the bentonite beds were detrimental to the tunnel progress. The beds were often situated above the tunnel roof, and the shale was not competent enough to hold an arch below the bentonite if the bed was 2 feet or less above the desired breakline. Sloughing to this zone of weakness increased overbreak.

When encountered, the main fossil horizon spalled in small pieces and required constant sealing until supported by timber and lagging. The 1-3 feet bentonite bed appeared more or less at tunnel level throughout the tunnel excavations. A typical tunnel profile is shown on plate 36. Simultaneous work in progress at the time of the pilot tunnel driving was that of the pilot shaft, headblocks and open cut tunnel excavation.

6.3.3 Headblock and Open Cut Tunnels. Weathered material was removed by clam. Subfirm and firm shale was removed by chipping, with some line drilling and blasting. The walls were supported by side and end plates at 4 foot centers. Open and closed corrugated sheeting was placed behind the structural steel to prevent sloughing. The weathered character of the material exposed by headblock excavation led to the conclusion that it would be more advantageous to excavate that area between the headblock and the portal wall by similar open cut work.

6.3.4 Main Tunnel Enlargement. By enlarging 8 feet of ground around the pilot tunnel, additional lateral faults were exposed. In some portions of the shale, unequal pressures had resulted in bending of the strata rather than in faulting. The zones of bending were found to be zones of weakness in the larger tunnel. Bentonite beds and the fossil horizon were planes of weakness as in the pilot tunnel. In spite of the fact that the location of all faults were known and the ground drilled and blasted accordingly, several unavoidable cave-ins did occur, due to the intersections of fault planes, fault planes and bentonite beds, or fault planes and joint planes lying in the upper sidewalls or above the roof. The cave-ins were due to large blocks of "working" ground moving along the planes of weakness into the excavated area below before supports could be installed. In the majority of these cases, sloughing stopped at not more than 7 feet beyond the prescribed lines of excavation. A major cave-in in No. 4 tunnel was occasioned by the intersection of three fault planes 17 feet above the tunnel roof, with the ground sloughing into the tunnel from this apex. Jointing affected

excavation for the main tunnel as it did pilot tunnel driving. A condition encountered in the main tunnel that impeded progress was the fractured and shattered area which had developed to a marked degree over the pilot tunnel. This ground had remained open for a considerable length of time. One to four feet of sloughed and loosely hanging material rested upon the pilot tunnel timbers and lagging. Should the face be opposite a pilot set, the set and blocking effectively supported the ground around the pilot tunnel. Ordinarily, the main tunnel breast was not opposite a pilot tunnel timber set. Such a condition required placement of temporary stulls, stringers or cribbing shortly after blasting to hold the ground in place and to protect the miners.

6.4 POWERHOUSES. The character of the foundation for the powerhouses is discussed in Chapter 9.

CHAPTER 7. - FOUNDATION TREATMENT

7.1 TUNNEL GROUTING. Plates 202 and 203 show typical tunnel cross sections. Observations made by a periscope through holes drilled in the tunnel lining showed that openings existed behind the blocking and corrugated sheets that had been placed back of the ring beams during excavations. The openings were sealed by grouting. To be sure that no voids or air pockets remained after grouting was completed, a standard 5-pattern layout for grout pipes was developed. The layout of the holes was determined by the location of the purlins and steel ring supports, and each ring of holes was drilled at intervals of 40 to 50 feet. Valves were installed on the vent pipes so that grout could not escape. The general procedure was to force grout into the two lower holes until venting was observed in the holes upstream and, finally, grout appearing in the top vent pipes would indicate that the section was completely filled.

7.1.1 Grouting was accomplished in two operations; first, a low pressure grout of cement, sand and water was forced through the pipes by means of a grout machine manufactured by the Union Iron Works. It consisted essentially of a horizontal circular tank in which a series of revolving paddles kept the mix in suspension. The materials were admitted through a door which opened inward in a dome-like opening at the top. Air pressure applied in the tank sealed the door and forced the grout through connection hoses to the grout pipe. The second operation was called "high pressure" grouting. In this operation a neat cement grout consisting of one part cement and 12 to 20 gallons of water was used. The material was pumped through the same pipes used for the low pressure grouting. The high pressure grouting was designed to fill any voids that might have been left due to shrinking, and also to fill the seams and faults existing in the shale in the immediate vicinity of the tunnels. The pressure used varied from 30 pounds to a maximum of 100 pounds depending on the amount of grout required to cover over the tunnels. No grouting programs were conducted for the spillway, embankment or powerhouse areas.

7.2 PERMANENT FOUNDATION DRAINS.

7.2.1 Spillway: Approximately 17 miles of drain tile were installed under the spillway channel floor. Steel seal strips were used as water stops across all construction joints to prevent seepage. Lateral and longitudinal tile drains were provided beneath each joint to collect any seepage that passed the steel water stop. Seepage is all collected into a central 18-inch drain. Regular inspections of the central drain have been made at about semi-yearly intervals and indications are that it is effective. There are 17 miles of gravel packed drains beneath the spillway channel floor and side walls which drain the faults under the spillway. The drains have been designed to intersect certain faults that appeared to be causing more than normal movement in the past. The drains are fitted with an elbow pointing downstream, so there will be no danger of introducing water into the drains during periods when the spillway is discharging. In the past, periods of maximum discharge from the drains seemed to be associated with movement across the "M" bentonite. The stratigraphic position of the "M" bentonite is shown on Plate 89. The horizontal drains are very effective indicators of the presence of water in the joints and faults in the shale underneath the spillway.

CHAPTER 8. - INSTRUMENTATION

8.1 MAIN EMBANKMENT. As previously discussed, the foundation materials in the valley adjacent to the dam, in descending order, originally consisted of a top stratum of impervious clay from 3 to 30 feet in depth (removed beneath the dam prior to construction); a sandy material approximately 20 feet in thickness; approximately 50 feet of clay; and a pervious stratum of sand and gravel that is 50 to 100 feet thick. The sand and gravel contain interspersed lenses of clay and silt. As previously discussed, a sheet steel pile cutoff wall was provided in the original plans. As an added precaution against percolation, the top of the cutoff wall was extended a minimum of 20 feet up into the hydraulic earth fill core of the dam to prevent excessive seepage along the junction of the fill and foundation materials. The sheet steel piling cutoff wall, as constructed, effectively reduced the percolation of water through the subsurface strata, but some percolation continued and considerable hydrostatic pressure developed in the deep pervious stratum under the downstream areas as the reservoir water surface elevation increased.

8.1.1 Piezometer Installation. During the years 1939 and 1940, numerous piezometer pipes were installed in the dam and in the foundation below the dam to observe hydrostatic conditions and to check the efficiency of the cut-off wall as the reservoir began to fill. The reservoir was kept at a very low level until 1942. When the reservoir began to fill in 1942, the pressure in the deep strata in the downstream area increased at a rapid rate. A test well at station 60+52, Range 18+66-D in the downstream area which had been installed during August 1937, and tapped the gravel layer beneath the overlying clay, showed artesian flow. In October 1939, visual observations indicated the well was flowing at an increased rate, and pressure observations were made on the well at various times after this date. As the reservoir filled rapidly in 1942, the flow from the well greatly increased.

Flow was measured at over one-half c.f.s. and carried a considerable amount of fines. Pressure in the downstream area was relieved by the installation of temporary wells. Piezometer pipes were then installed at 500-foot intervals along the downstream toe of the dam. Readings indicated that excessive pressures in the pervious stratum existed beneath the clay. A maximum pressure, equivalent to a head of 45 feet of water above the ground surface, was observed, and studies of piezometer observations indicated that, if the hydraulic pressure was not relieved, an equivalent head of over 70 feet at full reservoir elevation would exist. A board of consultants was convened, and after studying piezometer data, a system of relief wells was recommended near the downstream toe. The objective was to reduce the hydrostatic head in the downstream area to a value less than 15 feet above the ground surface. The permanent relief wells are discussed later.

As the dam neared completion, piezometers were installed in order to observe the hydrostatic pressure in the dam and foundation after the dam was completed and the reservoir impounded. The initial piezometers were installed in 1939 and 1940 at Stations 35+00 and 60+00. Other piezometers were installed in the east and west abutment areas and were utilized to investigate hydrostatic pressures in the shale foundation materials. They were not used in the study of underseepage. In 1942, the emergency relief well system was completed in the downstream area. Prior to drilling the pressure relief wells, a line of 4-inch diameter piezometers at 500-foot intervals was installed along range 19+50-D from Station 35+00 to Station 80+00 to fully investigate the hydrostatic pressures in this area. Piezometers were also installed along Station 70+00 at Range 1+60-D and at Ranges 3+00-D, 14+00-D and 19+50-D. The additional piezometers were located at the same ranges as earlier piezometers at Stations 35+00 and 60+00 which tapped the pervious material beneath the clay. Along Station 60+00 additional piezometers were installed downstream at 500-foot intervals to Range 50+00-D and tapped the same pervious material beneath the clay.

8.1.1.1 Permanent Piezometer Installations. Beginning in 1948 a program was started to replace the earlier piezometers. In addition, a program was authorized to salvage 14 of the old emergency wells along Range 20+00 and use them as permanent piezometers. The rest were plugged and backfilled. The program of salvaging or disposing of the emergency wells was completed in 1949. However, all the temporary wells were read as piezometers from the time the permanent wells were installed until they were taken out of operation. Piezometer locations are shown on Plates 38 through 50 and Plate 52.

In the program of replacing essential piezometer pipes some were abandoned. Others were replaced at locations where more data was needed. New piezometers were installed at Station 700+00, Range 30+00, and along Station 50+00 at the following Ranges: 1+25 U, 2+75-D and 30+00-D. These piezometers all tap the pervious stratum beneath the clay stratum, which is the same pervious strata tapped by the pressure relief wells.

8.1.2 Relief Wells: The initial spacing of the wells was 50 feet, but, the spacing was increased to 100 feet and then to 250 feet. Wells range in size from 6 inches to 10 inches and were installed by the Norbeck Drilling Company of Redfield, South Dakota with a completion date of September 1942. Initially there was no time to secure adequate material due to war-time shortages. Ordinary steel pipe which was on hand was used for screen and casing. As the wells were installed, the pressure in the area decreased and it was apparent that the system of wells, as installed, was sufficient for the time. A meeting of the Board of Consultants was called during the spring of 1943 to review the system after the initial period of stable operation. The Board of Consultants considered the system of relief wells, as installed, adequate for maximum reservoir conditions, but recommended that a new system of more permanent wells be planned to replace the original wells as soon as materials were available after the war. The temporary well system operated without trouble until June 1944, when one well failed.

8.1.2.1 Studies for Design of Permanent Well System: Upon the failure of the first temporary well (well No. 8), studies were initiated to determine the design of a more permanent system and to install new emergency wells. It was decided to use wood as the material for the permanent well system. Plate 64 shows the details of a well as finally designed for the permanent system. Relief well locations are shown on Plate 57 and Photo 35 shows the installation of a well screen.

8.1.2.2 Installation of Permanent Well System. The contract for the permanent relief well system was awarded to Joel Norling on 27 April 1946. Actual construction did not start until 14 June 1946, and the first well was completed on 31 August 1946. The entire well system was completed and accepted 4 December 1946. As the permanent wells were installed and put into service high flows stopped in the older, temporary wells. The remaining temporary wells were allowed to flow until April 1947 when they were all closed off and used as piezometers. The location of permanent embankment relief wells is shown on plate 62. The collector system for the wells was not completed until 1949. The permanent pressure relief wells have functioned without difficulties from the time they were installed and the total discharge from the well system has steadily decreased. Regular readings have been taken of the pressure relief wells and related piezometer pipes since installation and will continue in the future.

8.1.3 Piezometer and Pressure Relief Well Observations. As the installation of the emergency relief wells progressed, piezometer readings were made daily, and after a "test period" in November 1942, all the piezometers relating to the pressure relief well system were read at regular weekly intervals. The standard discharge elevation of the permanent relief wells was 2047 m.s.l. However, some head was required to discharge through the orifice weirs, so the wells had a somewhat higher head than if they could have discharged freely at this elevation.

The total discharge of the pressure relief well system has always been computed by summation of the discharges from the individual wells. There have never been fewer than 17 wells, and there have been as many as 24 wells operating at one time. A summary of discharge data for the downstream relief well system is shown on plates 66 through 74. Discharge data for the left abutment relief wells is given on plates 75 through 79 and data for the downstream toe area is shown on plates 80 through 84.

8.1.4 Operation of Piezometers Prior to 1942. Prior to 1942, the piezometers which tapped the pervious material beneath the clay did not operate freely, and in most cases it was only at certain times when a reversal of pressure took place that a value could be obtained which was close to the true hydrostatic pressure reading. Even though the response of most of the pipes was slow prior to 1942, a study of the data obtained from them has been useful in the past.

8.1.5 Operation of Piezometers Since 1942. The piezometers at stations 35+00, 60+00 and 70+00 were jetted and tested before the series of pressure relief wells was installed in October 1942. Since that time, all of the piezometers have been operating satisfactorily. Main embankment piezometer locations are shown on plates 40, 41, 45, 47, 48, 49 and 57.

8.1.6 Downstream Relief Wells. A report titled "Report on Improvement of Relief Well Design, Fort Peck Dam, Missouri River, Montana," was published in June 1951. The report covers the history, design, and operation of temporary and permanent pressure relief well systems up to the date of publication and should be consulted for details concerning the wells. A memorandum report, dated November 1950, titled "Report on Fort Peck Relief Well Discharge and Related Data" covers the discharge observation history of the pressure relief wells from the initial installation to date of the report and should be consulted for details on measuring devices used to check the well discharge and for problems encountered. All available data concerning the excess substratum pressure at the downstream toe was reviewed and a report was submitted to the Division Engineer, dated 3 September 1942, subject: "Inspection of Pressure Relief Well System at Downstream Toe of Fort Peck

Dam, Fort Peck, Montana." This report was quite comprehensive and correlated all available data up to the date of the report. Hydrostatic pressure readings in foundation clays and gravel layers are shown on plates 38 through 41 and 45 through 49. Observations and tests have been made on all wells at regular intervals since installation. Yearly graphs of observations have been summarized in the plots as shown on plates 65 through 75. The plots indicate significant trends and present the best overall picture of the performance of the relief wells with respect to all factors. The discharge from the relief wells has gradually decreased with time since 1943. Indications are that there has been an overall reduction of flow with time, and the reduction of flow has been at a faster rate in the upper gravel than in the lower. The piezometric surface during the various time periods is summarized by plates 38 through 41 and plates 45 through 49. Plate 38 summarizes the piezometric surface within the embankment prior to installation of the relief wells, during relief well testing, and after installation of the relief well The piezometric surface is represented by pressure head on the above mentioned plates.

8.1.7 Left Abutment Relief Wells. Seepage appeared in the left abutment on 20 February 1945. The point of seepage was at the edge of the abutment and just downstream from the mouth of Coulee "B" at Range 29+90-D. A piezometer pipe was installed at Station 38+00, Range 29+50-D, immediately upstream from where the seepage had first appeared. The piezometer penetrated through semipervious surface material to tap an underlying layer of clean pervious sand. There was an appreciable flow from the piezometer, which was allowed to discharge in order to provide as much pressure relief as possible. The discharge from the piezometer pipe and the seepage from the wet area continued to increase during the next two years, and the ground was soft and unstable for a considerable area along the edge of the abutment. Remedial measures consisted of a system of pressure relief wells that discharged into an underground collector pipe. The work was accomplished by Government Plant and Hired Labor. Locations of left abutment relief wells are shown on plate 57.

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8.1.7 Left Abutment Relief Wells. Seepage appeared in the left abutment on 20 February 1945. The point of seepage was at the edge of the abutment and just downstream from the mouth of Coulee "B" at Range 29+90-D. A piezometer pipe was installed at Station 88+00, Range 29+50-D, immediately upstream from where the seepage had first appeared. The piezometer penetrated through semipervious surface material to tap an underlying layer of clean pervious sand. There was an appreciable flow from the piezometer, which was allowed to discharge in order to provide as much pressure relief as possible. The discharge from the piezometer pipe and the seepage from the wet area continued to increase during the next two years, and the ground was soft and unstable for a considerable area along the edge of the abutment. Remedial measures consisted of a system of pressure relief wells that discharged into an underground collector pipe. The work was accomplished by Government Plant and Hired Labor. Locations of left abutment relief wells are shown on plate 57.

8.1.7.1 Design of the Well System. In the design of the well system, the physical location of the wells was governed largely by topography. Over most of the area it was impossible to tap the pervious strata along the abutment with vertical wells except at the mouth of coulees where the wells could be located back of the abutment line. The seepage water finding its way through the pervious zone to the cutoff at the edge of the abutment was drained by relief wells discharging into an underground collector. The well system was designed to lower the hydrostatic pressure in the seepage areas to a point below the ground surface.

8.1.7.1.1 The area along the edge of the abutment where wells could be effective was limited. This in turn, limited the number of wells. Any appreciable increase in discharge would tend to overtax individual wells and raise the saturation line. In order to prevent or limit the increase of seepage to the edge of the abutment, it was decided to tap the pervious strata with additional wells upstream from the abutment in coulees where significant drawdowns could be achieved. The wells in the coulees upstream from the abutment were spaced on approximately 400-foot centers. The wells along the edge of the abutment at Coulee "B" were spaced on 50-foot centers in order to properly drain this area. The wells in Coulee "D" were spaced on 100-foot centers and an additional well was placed at the mouth of Coulee "C" to relieve a point of seepage which appeared after construction was started. Originally, 13 wells were included in the plans, but before the installation was completed, three more wells were authorized to take care of additional seepage. Several years after the initial installation was completed and in operation, additional seepage in Coulee "A" became evident. This phase of the left abutment installation is discussed later under Coulee "A."

8.1.7.1.2 Creosote-treated wood pipe was used for the screen and riser (photo 35). A transition from wood to metal was made at a depth below the ground surface where the soil was permanently moist. The water discharged by the wells was corrosive so brass was used above the transition section and for all surface pipe, fittings, and valves. Design studies also indicated that smaller wells at closer spacings would be more effective than larger

wells at wider spacings. The pervious material to be drained contained sufficient fines to require the use of a gravel pack with the wood screen section. The outside diameter of the wood pipe was approximately 5 inches. A 10-inch pipe was used for casing the hole, which allowed a 2-1/2-inch envelope of gravel pack around the screen.

8.1.7.2 Installation of Well System. The actual drilling and installation of the left abutment pressure relief wells were accomplished by Government Plant and Hired Labor. The wells were drilled with a 12-inch hydraulic spade jet and mud was used as a drilling fluid. As soon as the pervious material was encountered, 10-inch casing was installed and the hole was drilled by drive sample methods to shale. The 10-inch casing was sealed into the shale and the drill mud was then jetted out of the pipe and replaced with clear water prior to the installation of the wood pipe. For a number of the wells the hydrostatic head exceeded 30 feet above the ground surface at the time the well was drilled. After the installation of the wood casing and screens, the gravel pack was added as described and mud was pumped in the casing on both the inside and outside of the wood pipe. Several lengths of 2-inch extra heavy pipe were inserted inside of the wood pipe to hold it down. A careful watch was kept during jacking operations, and if the wood casing showed a tendency to move, it was pounded lightly while the jacking was going on.

8.1.7.3 Coulee "A" Drainage. The installation of the left abutment underseepage system was started on 16 June 1947 and was completed on 1 July 1948. During the investigations for the left abutment well system several test holes were augered in Coulee "A." The test holes indicated that the saturation line was below the ground surface and the material above the shale was a semipervious clay loam. After the 1947-48 installation of the relief well system in adjacent coulees, the entire abutment area dried up and was stable. During late 1950 and 1951 there was a noticable increase of water levels in several of the piezometer pipes near Coulee "A" and a corresponding increase in the well discharge of well Nos. 1 and 2 in Coulee "B." Also in 1951, a slide occurred along the edge of the abutment on the knoll forming the divide between Coulees "A" and "B." An investigation of the slide and the adjacent area near the mouth of Coulee "A" was initiated, and it was

found that the material underlying the slide was saturated. The saturated area extended up Coulee "A" for a considerable distance along the base of the dividing ridge between Coulees "A" and "B." Saturation conditions, as found at the time, created a boggy and unstable condition in the lower portion of Coulee "A." In the fall of 1951 several test holes were drilled in the Coulee and the immediate vicinity of the Coulee. Drill logs indicated that the shale bedrock rose abruptly in elevation at a point approximately 165 feet back from the mouth of the Coulee. The seepage in Coulee A occurs downstream from a high shale bedrock area (previously mentioned in this report as a shale "island".) The area of high bedrock could funnel any seepage around the left abutment into an area from range 23+00-D to range 37+00-D. Coulee "A" is located within this area. Plate 42 shows the relationship of Coulee "A" to the bedrock high and plate 12 shows the Coulee in profile. Prior to high reservoir elevations, areas downstream from the dike had been soggy from water draining from the hydraulically placed core of the dike. Seepage from Coulee "A" could have originated from fluctuations in reservoir levels. As the levels increase, the cores of the dike and embankment were resaturated. As the reservoir lowered, the cores drained and the drainage was funneled around the left abutment bedrock high into Coulee "A".

8.1.7.3.1 A method of horizontal drains was adopted as being the most feasible and least expensive for correcting the seepage problem. Authority was granted to install a horizontal plastic drain and collector system by Government Plant and Hired Labor. The horizontal plastic drain was not able to lower the saturation line below the ground surface in Coulee "A" and it was decided not to install additional drains of this type. Consequently, a 24-foot long slotted wood pipe was installed in a trench. The wood pipe was set at a depth of approximately 8 feet below the existing ground surface and 4 to 6 feet above the shale bedrock. The installation of the wood screen improved drainage of the area. However, piezometer observations disclosed that the system was still not providing sufficient pressure relief to bring about an acceptable lowering of the saturation line. Apparently the seepage water was transmitted to the immediate area of Coulee "A" through pervious material, but there was only semipervious material present in the area where it was possible to install drainage facilities.

8.1.7.3.2 Further study was made of the problem, and a gravel-packed drain tile through the remaining wet and boggy area was installed. To make the drain tile more effective, a series of vertical drains were installed from the bottom of the trench to intersect all pervious strata existing between the top of the shale and the bottom of the trench. The vertical drains were holes drilled to shale and backfilled with graded gravel. The vertical drains were installed during the fall of 1953. The installation of the final drain corrected the unstable and boggy conditions.

8.1.8 Results of Observations - Left Abutment. The initial seepage discharge from the left abutment started on 20 February 1945. Discharge measurements were made at infrequent intervals thereafter in order to have a rough check on the amount of water passing through the abutment. The left abutment wells were installed during 1947 and 1948, and as each well was installed, periodic discharge observations were made. Upon the completion of the well system, all seepage was controlled and could be accurately measured. Daily discharge measurements were made during the construction period. The permanent chronological records for each well are on file at the District office and these records, along with the left abutment piezometer records, furnish complete data on the hydraulic conditions of the left abutment at all times. The graphs shown on plates 75 through 78 indicate the hydrostatic pressure and total discharge from the abutment since 1970.

8.1.9 Seepage Pipes. The original seepage pipes in the main embankment were dual purpose installations that were used for foundation subsidence observations as well as checking the saturation line in the dam. A 3-inch, perforated iron pipe was installed on top of an 8'x8' plank platform at the base of the dam and, as the fill was raised, pipe was added. The iron pipe was perforated by drilling 1/16-inch holes spirally around the pipe at intervals of six holes per foot. The pipes were designed to read the average saturation line or phreatic line in the main fill. The original pipes served the intended purpose during the major construction of the dam. In 1938, it was determined that seepage pipes should be a permanent feature of the dam, and

plans were made for a permanent system. Four-inch perforated iron pipes were carefully sand blasted clean and painted with a special protective enamel. The pipes were installed by Government Plant and Hired Labor, using a churn drill and the drive sample technique of drilling. The pipes have survived with only a minimum of trouble since installation. The water in the pipe tends to get rusty at times and plug the perforations, but this is removed by periodic jetting or bailing. In 1943, a number of seepage pipes were placed in the left abutment to check the saturation line. The installations consist of 2-1/2-inch perforated, galvanized-iron pipe installed in a 6-inch hole and backfilled with clean concrete sand. In 1947, additional seepage pipes were installed on the downstream slope of the dike at Stations 115+00, 145+00 and 165+00. The installations were made to monitor the saturation line through the dike section and through the damp area downstream from the dike.

8.1.9.1 Seepage pipes were read at frequent intervals during early construction phases of the dam, and each year a seepage report was prepared which summarized the observations. The early reports are on file in the Omaha District office. It was found that after the hydraulic fill was completed the changes in the saturation line occurred at a very slow rate, and in 1944 the annual seepage reports were discontinued.

8.1.9.1.1 Periodic monthly observations are taken on approximately 60 seepage pipes at the following stations on the main dam: 10+00, 15+00, 20+00, 35+00, 50+00, 67+00, 81+00 and 90+00. In addition, there are 13 seepage pipe installations which cover the left abutment area at the following dike stations: 115+00, 145+00, 165+00 and 205+00. The continued observation of the above pipes at regular intervals furnishes data on the saturation line in all areas of the main dam, the dike, and the immediate area downstream from the dike. Periodic seepage pipe observations provide the necessary monitoring of the dike area whenever the reservoir is held at a consistently high elevation (exceeding 2230 m.s.l.) over an extended period of time.

8.1.10 Settlement and Subsidence Instrumentation. The terms settlement and subsidence were adopted at Fort Peck as names for certain types of pipes used for measuring vertical movements. Subsidence pipe was the name adopted in 1935 for the pipe installations at the base of the dam for the purpose of measuring foundation consolidation (or subsidence) due to the load imposed by the embankment. The term settlement pipe was adopted for the pipe installations in the embankment for the purpose of measuring total settlement. Total settlement includes foundation consolidation plus embankment consolidation. Settlement pipes were usually shallow installations installed just prior to the completion of the embankment. Both types of installations were made at Fort Peck and they have furnished data on the combined settlement and subsidence of the Fort Peck embankment and the embankment's foundation. However, it is not possible to determine embankment consolidation. The subsidence pipe installations were made at Fort Peck to collect data on the behavior of the foundation as it was loaded. Each installation consisted of a 3-inch pipe welded to a flat steel plate 18 inches square, and this, in turn, was bolted to a wood base that was 8 feet square and made up of three layers of plank laid alternately at right angles. The wood base with the initial length of pipe was placed at a designated location on the foundation, and levels were run to the top of pipe. The distance from the top of the pipe to the base was measured and the initial elevation of the wood base was computed and recorded. As the fill progressed, additional measured lengths of pipe were added, and all additions were recorded. Levels were run to the subsidence pipe at intervals as the fill progressed, and the elevation of the wood base was computed and recorded each time. There was no provision for a protector pipe around the subsidence pipe; consequently, the data reflects some strain due to skin friction of the embankment soil around the pipe. In practice it was difficult to preserve the pipe installations during construction activities, and most of the core subsidence pipes were lost before completion of the dam. Some pipes were off plumb, and special measurements and calculations had to be made to compute the true elevation of the mat. There are currently 35 settlement and subsidence pipes in operation along the axis of the main embankment and dike. Twelve pipes are in operation along the downstream toe of the embankment.

Temporary settlement pipes were installed as the embankment neared completion to measure the rate of total settlement. As the topping out of the dam neared completion, the temporary installation was replaced by permanent settlement points which consisted of a 2-inch pipe placed in an auger hole about 6 feet deep and driven into the fill until solid. The settlement pipe was protected with a 3-inch outer casing that extended to the ground surface. The settlement pipes were established on a definite range line so they could be marked and used for subsequent horizontal control as well as for vertical control. The settlement pipes installed in 1940 were all carefully extended when the fill was raised to its final height in the topping out operation in 1946.

8.1.10.1 In 1941, additional settlement pipes were installed along the crest of the dike. The additional pipes were not set at any specific range location, and the station location of each pipe was only approximate. These pipes were also extended when the dam was raised to final grade in 1946. The settlement pipes across the top of the main dam and dike section form a continuous run of approximately four miles, and extend from the bench mark in Emergency Control Shaft No. 4 (at about Station 1+00) to bench mark No. 5 (at approximate Station 205+00).

8.1.10.2 As discussed previously, the hydrostatic uplift pressure in the area downstream from the dam increased to a value equivalent to 45 feet of head above the ground surface in the year 1942. To determine the effect on vertical movement, settlement pipes were established at 500-foot intervals across the entire valley adjacent to the toe of the dam along Range 19+00. In addition, movement points were welded to each piezometer pipe located at 500-foot intervals along Station 60+00 from the downstream toe to Range 50+00-D. The line of settlement points was at right angles to the line across the toe, and was tied to a permanent bench mark (BM Garden). Plates 14 through 20 show the location of all settlement and subsidence installations and show foundation settlement beneath the embankment.

8.1.11 Foundation Subsidence Observations. Initial observations of subsidence pipes began as soon as the first wood base was set and its elevation determined. The pipes were checked at frequent intervals, especially during the construction season when the fill was being raised rapidly.

8.1.11.1 After the fill was completed, the frequency of observations was reduced to twice a year until 1951 when it was reduced to once per year. In 1954 the frequency was changed to once every 2 years. The rate of settlement has been decreasing with time, and longer periods of time are now necessary to show any significant changes. Regular observations are made every 2 years. All subsidence observations are recorded in permanent record files which consist of a tabulated chronological history for each installation. The record is kept in the working files at the District office, and the original survey notes for all data are on file at the Area office in Fort Peck. During, and immediately after, construction, graphical records were kept of all subsidence data. The graphs are also on file at the District office. Initial graphs were of value during construction to show settlement as the embankment load was added. However, after the embankment was completed, the rate of settlement or consolidation continued at a slower rate. In order to maintain a check on the rate of settlement, time settlement graphs on semi-log paper have been prepared. The data plotted on the semi-log graphs has followed a definite trend up to the present. Plate 17 shows a graphical summary of the foundation subsidence at Station 60+00 of the main embankment, together with generalized foundation conditions. Maximum subsidence of the embankment's foundation has been reported as 16 feet for a maximum fill height of 250 feet. Since all subsidence installations under the core fill section were destroyed during construction, figures for maximum foundation subsidence must of necessity be computed from data obtained from adjacent installations where foundation conditions are similar. The subsidence data for Stations 50, 60 and 67 (Plates 16 through 18) indicate that subsidence prior to 1957 averaged from .062 to .068 feet

per foot of fill height, for a maximum fill height of 250.5 feet; this results in a computed foundation subsidence of 15.5 to 17 feet. Using fill heights, adjusted for foundation settlement during construction, the maximum foundation subsidence in the old river channel (Station 60+00) could be about 17.4 feet (using average settlement value of 0.065 feet per foot of fill height at this station). Total embankment settlement is not known.

8.2 Information on instrumentation for the powerhouses and the spillway is discussed in Chapter 9.

CHAPTER 9. - SPECIAL FOUNDATION PROBLEMS

9.1 SLIDE - MAIN EMBANKMENT, EAST ABUTMENT: On the morning of September 22, 1938, cracks appeared on the upstream face of the main embankment as portions of the upstream shell began to slide into the core pool. Simultaneously, the main mass, hinged on the right (east) abutment, moved out into the reservoir. The cleavage line where the west end of the mass broke loose was at station 27+00. A total of 5,217,000 cubic yards moved 1,200 ft. horizontally over a period of 10 minutes and resulted in a rupture in the upstream shell 1,700 ft. long. Of the 180 workers in the area, 34 were carried along with the moving earth mass and eight of these men lost their lives. Six of the eight men were never found. Photo No. 1 is an aerial view of the slide.

9.1.1 A meeting of the board of consultants was called to evaluate causes of the slide and to recommend changes in embankment design.

9.1.2 Extensive drilling of 2-inch and 6-inch bore holes, Calyx holes and shafts indicated that a portion of the shale bluff on the abutment area had failed along a fault plane (designated the "A" fault). It was found that water had penetrated along bedding planes of weathered shale on the abutment, particularly following bentonite beds, to a point equivalent to 30 feet in depth. The upstream shell of the embankment had moved en masse carrying portions of the shale on the abutment with it.

9.1.3 Explorations that were made subsequent to the slide indicated distinct shear planes on, or in, weathered portions of the shale foundation on the abutment and in bentonite seams within the shale. The bentonite seams had a maximum thickness of 15 inches.

After consideration of all of the data, the board of consultants concluded: "...the slide in the upstream portion of the dam near the right abutment was due to the fact that the shearing resistance of the weathered shale and

bentonite seams in the abutment foundation was insufficient to withstand the shearing force to which the foundation was subjected, and the extent to which the slide progressed upstream may have been due, in some degree, to a partial liquification of the fill material in the slide."

9.1.4 The board of consultants went on to recommend the following:

(1) Bond the remaining old (pre-slide) core of the embankment to new core material by means of a single row of sheet piling.

(2) Installation of piezometers in the shale on both abutments for the purpose of determining hydrostatic pressures and the installation of drainage wells where necessary.

(3) Since the reconstructed embankment area would have flatter slopes, the board did not feel it necessary to strip weathered shale from the abutment since shearing stress would be of low intensity, with ample factors of safety, on the weathered shale when graded to lower slopes.

(4) The Board approved a plan of the District Engineer to study further the possibility of grouting fault zones in the shale on the east abutment, but this proposed grouting program was not done.

(5) The Board concurred with the District Engineer's plan to flatten slopes behind the intake structure.

9.1.5 The slide was of such magnitude that it increased construction time for the main embankment by two years.

9.2 SLIDE PROBLEMS, POWERHOUSE SLOPE: Since 1934, when construction began on the outlet works, sliding was a problem on the powerhouse slope which consists of a row of hills directly north of the powerhouse. The slides were

a nuisance and a threat to the switchyards and paved access road. Soon after the start of construction for the powerhouse outlet works began, a small slide occurred in June 1934 and was followed, a few weeks later, by a larger slide. After the second slide, the grading plan for slopes above the powerhouse outlet works was revised to provide 1V on 3H backslopes and work began with removal of material from above the 1V on 3H slope. Soon after this, a more extensive slide (Slide #3) occurred. A fourth slide occurred on 21 September, 1934 and was quite extensive. Slide #4 coincided approximately with the old slide scarp (Slide #1). It is estimated that the major movement of the fourth slide initiated subsequent slides #5 and #6 which represented additional movements of the slide mass with upstream extensions of the scarp. Plate 160 shows the 1934 and 1970 positions of the scarp line. After completion of the outlet works, movement continued on the powerhouse slope but there is very little factual record to substantiate the movement. After completion of Powerhouse No. 1, the slopes were kept in a smooth well graded condition with numerous surface drainage ditches. That movement continued to occur is verified by the fact that it was necessary to remove material from the road ditch at the toe of the slope on 9 July 1943, 10 December 1943, and 31 August 1944. During this time period, a line of iron pipes was installed as survey points. Survey checks on this line from 9 July, 1944 to 10 September 1945 indicated that the slide mass moved about four feet per year.

9.2.1 Foundation investigations for Powerhouse No. 2 included an investigation of the powerhouse slope to determine if slide movement would endanger the foundation excavation. This investigation included the drilling of eleven exploratory holes on the slope adjacent to the powerhouse and the installation of piezometers in all deep holes. In addition, four tiltmeters were installed in back of the outlet walls and on the slope opposite the foundation excavation area. The results of slope movement monitoring efforts prior to 1972 are summarized by the movement contours on plate 156. Movement from 1977 to 1982 is shown by plate 160I.

9.2.2 After the initial resurvey of all pipes in the slide area, temporary iron pipes were established as chaining points for making subsequent resur-

veys. The points furnished additional data on the slide movement. During 1971, two additional pipe lines were established to check for movement outside the active slide area. The plan location and tabular results of movement data are shown on plates 157 and 171.

9.2.3 During the summer of 1970, the outlet portal road and a storm drain were constructed at the base of the slide. Considerable material was removed from the toe of the powerhouse slope and movement occurred at the downstream portion of the outlet works slope. The removal of toe material probably accelerated the movement of the active slide. The toe of the slide scarp was very apparent and small amounts of movement were very noticeable. The toe of the slide scarp was mapped and gages were designed to monitor movement across the toe of the slide. Plate 160 shows the locations of a dial gage, five tension wire gages, and the toe of the slide scarp. Also shown are the outline of the original slide and the slide scarp as surveyed in 1970.

9.2.4 A dial gage was installed the farthest downstream of the instruments and just upstream of tunnel headblock #4 (Plate 160). From mid-December 1970, through March 1972, this gage showed movement of about 2.3 inches per year. After March, movement ceased but resumed about October 1971. The average rate of movement from October, 1971 to March, 1972 was about 0.4 of an inch per year.

9.2.5 Wire gage #1 was located above the switchyard (plate 160). The rate of movement for the first year (November 1970 - November 1971) was 16 inches. Rates of movement declined through February 1972 but increased again in early March 1972. Wire gage #2 (plate 160) showed a rate of movement for the first year of 18-1/2 inches. Wire gage 3 was installed to measure differential movement across major cracks in the slide area. Negative readings indicated that the downslope shale mass was moving faster than the upslope mass. Gage 4 measured movement across the upstream limits of the main slide area. Gage 5 was installed across the estimated upstream edge of the slide mass. Plates 74A and 74B show movement records of the gages prior to Phase I and II slope excavation (movement up to 1972) for all gages.

9.2.6 In 1971, two tiltmeters were installed to check on deep seated movement in the powerhouse slope area. The individual logs of these holes are shown on plates 162 and 163. The tiltmeter profile is shown on plate 163, and plates 166 and 167 show tiltmeter locations. The movement record of tiltmeter WS-6 observations is shown on plate 167. Readings indicate that there was no significant movement in this area. Tiltmeter WS-7 was installed to check on any possible movement which would effect the walls of the outlet works or for possible deep seated movement in this area adjacent to the powerhouse. Tiltmeter WS-7 is shown on the P-1 thru P-5 profile on plate 162. The record of observations for WS-7 is shown on plate 167.

9.2.7 The original tiltmeter installation on the powerhouse slope was made during February 1957. The significant results of the observations are shown on plates 166 through 170. Plate 166 shows movement records for WS-3 and WS-4. WS-3 was installed 13 February, 1957. The movement record for WS-3 indicated a narrow movement zone between 54.0 - 56.0 foot depths. The resultant rate of movement was about 2 inches per year. WS-4 was installed on 7 February 1957. Movement showed up in the north-south set of grooves. No significant movement was indicated in the east-west direction. The major movement zone was between the 56.5 and 59.0 foot depth zone. The rate of movement was about 3.2 inches per year.

9.2.8 Plate 166 also shows the movement record for WS-1 and WS-2 and indicates movement between the 12.5 and 16.0 foot depth as last observed on 29 April 1959. The average rate of movement for a 2.13 year period was 0.37 inches per year. WS-2 sheared off at the 12.5 foot depth zone. WS-1 indicated no significant movement when last observed on 21 October, 1958.

9.2.9 Piezometers were installed to measure the hydrostatic pressure in the suspected slide plane and to locate the slide planes through the use of weak couplings on the piezometers. Observation records and locations of the piezometers are indicated on plates 168, 169, and 170. By July 1972, some piezometers had moved 40 feet down slope from their original position and tops-of-pipe often had been lowered several feet. Significant rises in water level were noted in piezometers P-3V, P-10V, and P-11V. In general, water

level measurements in the slide area seemed to indicate water in, and above, the slide plane. Several of the deep piezometers such as 10A, 9V, 8U, and WS-2 indicated much lower water levels. In fact, piezometer 10A indicated a water level that was below the tailwater elevation. However, piezometer 10-V, which was only a few feet away from 10A showed water levels that were about 100 feet higher than 10A. The water level used for stability analysis was the water level in the slide plane. Bore hole information, geologic profiles and slide plane interpretations are given on plates 161 through 165.

9.2.10 In May 1973, a contract was awarded to William Clairmont, Inc. of Bismarck, North Dakota to reduce the powerhouse slope to 1V on 6H. From 23 June to 19 September 1973, geologic conditions of the slide area were mapped by Omaha District's geology section as subsurface conditions were exposed by grading operations. Plates 160A - 160H show the results of field mapping. The mapping indicated the presence of a graben, or downthrown block, between stations 27+40 and 28+70 west of the old 115 KV hill. The two faults which bound the graben appear to roughly strike in a direction between north - south and North 25° East. The fault on the upstream side dips vertically. Vertical displacement of 20 feet along the fault was determined by offset bentonites. Bentonite horizons are shown on the log of P-10A (plate 160G). In borehole P-10A, a complete column of bentonites on the powerhouse slope is represented. The fault on the downstream side of the graben was not exposed but was inferred by the sequence of bentonite layers. West of Station 28+70 the L + 10 pair, M, M + 1, N and O bentonites were found in a haul road cut at a higher elevation than the P + 1 bentonite along the above mentioned fault. Since strata in this area dip anywhere from 7° to 65° to the northwest, it was not possible to determine how much displacement was represented on the unseen downstream fault, or fault zone. However, it was apparent that the downstream block, west of Station 28+70, is stratigraphically higher than the upstream block, which is east of Station 27+55 and is considerably higher than the graben. Further evidence for the presence of a graben is an old gulley that occupied the area before it was filled in 1934 (Photo 16).

9.2.11 The most obvious structural characteristic of the Bearpaw shale in the powerhouse slope area is that of slumping. The term slump is used to refer to the overall year-to-year movement of the slope. Slumps in the powerhouse area showed backward rotation and relatively slow rates of movement. Bentonite layers, especially the "M" and "P" played a major role in the slumping by serving as slide planes for the strata. The bentonite sequence between the P + 1 and M + 1 layers was found repeated between elevations 2,177 and 2,115. In some locations, unweathered shale was found above weathered shale as a result of sliding along a bentonite, but more commonly, weathered shale was found in sharp contact above less weathered or unweathered shale. Field mapping revealed the bedding trend of the bentonites (plates 160B, 160C, and 160D).

9.2.12 Geologic profiles on plates 160B, 160C, and 160D indicate that faults related to slumping extended at least as deep as elevation 2,000, or 150 feet beneath the original ground surface, and had vertical displacements at least as great as 40 to 45 feet. The greatest amount of vertical displacement that was measured during grading of the powerhouse slopes was 11 feet at Station 18+78, 318 left. It was felt that offsets due to faulting were the result of numerous smaller, nearly parallel, shears rather than a single slide plane. Slump bodies were found to be bounded upslope by moderate to steep faults that dipped 35° to 60° to the southwest and southeast. The faults were expressed as scarps on the original ground surface. The slump blocks are bounded laterally by faults which strike toward the river.

9.2.13 Slickenside altitudes obtained from field mapping indicated that the rakes of the slickensides trend to the southeast and southwest in a downslope direction.

9.2.14 It is believed that most faults in bedrock are related to slumping rather than older deep seated faults. The assumption is based on the fact that the Missouri River has cut overly-steep slopes into the right abutment. The slump structure on the powerhouse slope area was found to be very complex with slumps within larger slumps. Larger slump bodies were difficult to

recognize when mapped one exposure at a time because large amounts of shale, which appeared undisturbed, were included in the blocks. One of the largest slump bodies was delineated by the arc-like feature shown on plate 160E. This zone can be traced from Station 19+50, 325 lt. on the downstream side, to 13+14, 280 lt. at the center, to about 7+90, 265 lt. on the upstream side, and could be connected to the slump structure as far upstream as 5+00, 250 lt. In some locations the slump bodies appeared to form sharp boundaries with the less disturbed shale, but in most areas the slump boundaries also appeared to be gradational, with bedding nearly parallel on both sides of the slide planes. Away from the slump body the bedding gradually flattens and the shale is increasingly competent. Bedding on the flanks of the slump body tend to dip toward the middle of the slump mass since the central portion moved farther downslope than the side portions.

9.2.15 After operations began to reduce the slope during the latter part of May 1973, three distinct slides occured on the powerhouse slope. The first and largest slide began on 22 June 1973 between Stations 10+14 and 19+00 and resulted in large cracks on the first day. The slide outline is shown by plate 160E. As of 27 June 1973, about five feet of movement had taken place. The area had received 1.75 inches of rain in twelve hours from 17 June to 18 June. There also had been a thunderstorm on the night of 20 June. The second slide occurred on the afternoon of 11 July 1973 between Stations 3+00 and 6+50. Much of the area was covered with recent fill and large cracks and a partial headwall scarp were conspicuous. A third slide started early on the morning of 2 August 1973 between Stations 23+50 and 27+30 as Euclid wagons began to use a new haul road. This slide was on an old fill slope.

9.2.16 Slump planes and shallow faults on the powerhouse slope were felt to control ground water conditions. A perched water table existed in and above the low angle slump slide plane. In tiltmeter WS-4, the water level was about 23 feet above the slide plane as of September 1971. This water level could be a reflection of a higher subsidiary slide plane whose presence was indicated by well developed slickensides that were mapped during slope excavation.

9.2.17 Powerhouse slopes were laid back to final grades as shown on plate 155.

Dewatering during grading of the slope was carried out by the Corps of Engineers between Station 11+65, 445 Lt. and 12+30, 415 Lt. by five submersible pumps placed in the vicinity of the suspected slide plane. Ten wells were originally drilled in December 1972 and the approximate slide plane elevation was determined in each well by noting the depth of unweathered shale. From 31 January 1973 to 1 August 1973, a total of 34,900 gallons of water was pumped from the drill holes.

Wells located where the slide plane was lowest pumped most of the water.

9.2.18 Since the powerhouse slope was unloaded by grading operations, instrumentation has indicated negligible slope motion. For the record, Tiltmeter WS-6, plate 152 is intended to record movement at depths ranging from 101 feet to 111 feet within a badly fractured zone. The tiltmeter's record shows total movement of 0.3 of an inch toward the powerhouse and 0.7 of an inch in an upstream direction. The motion toward the powerhouse has essentially stopped and upstream uniform movement of 0.2 inch has continued since 1975. Powerhouse slope movements from 1977 to 1982 are summarized on plate 160I.

9.3 <u>FOUNDATION PROBLEMS, SPILLWAY AREA</u>: Movement observations have been made on the Fort Peck Spillway since early construction phases of the project. Early, during construction, it became apparent that movement was taking place. In May 1936, a benchmark, F6-b, was established on the gate structure and this benchmark with its established elevation of 2224.943 msl was used as the reference datum for construction of the entire spillway structure after this date. This same benchmark with its established elevation has also been used as the reference datum for all movement surveys. Movement data thus show movement relative to benchmark F6-b rather than movement with respect to MSL. Benchmark F6-b has itself been subject to movement since it was established.

Surveys indicated that the entire gate structure of the spillway had risen about 0.2 foot prior to 1944. Since 1944, the gate structure has been stable.

The true picture of spillway movement has been somewhat obscured by the fact that earlier recorded movements were differential movement based on a reference datum on the gate structure, which in itself had been subject to some movement. During the early years of movement observations, the movement of the bench mark tended to nullify the rebound effect when the area was unloaded. The early movement that occured in the spillway was probably the cumulative result of several causes. Many of these were recognized and reported at about the time the spillway was completed.

9.3.1 The high shale side slopes resulting from spillway excavation have, from the start, posed problems in slope stability (Photos 21, 22, 23, & 24). Numerous slides occurred during construction and since completion. No major slides have occurred since completion but there has been continued movement of the channel floor and side walls. The excessive movement at certain locations was believed to be the result of incipient slide movement with the movement concentrated in certain areas due to the intersection of fault lines. The concrete channel and side walls have acted as a toe load of considerable magnitude due to the fact that the entire channel floor and wall are dowelled together with reinforcing steel.

The shape of the movement pattern in the upper portion of the spillway channel where there has been a zone of maximum movement adjacent to each side seems to be an example of plastic deformation due to the surcharge of the high side slopes.

9.3.2 Spillway movement has been shown in summary contour form since regular observations began in 1939. Plates 92 and 101 show spillway movement prior to November 1960. The contour pattern has remained essentially the same, except for magnitude of movement, since 1943 (in some areas since 1939). Time movement graphs up to 1962 or 1965 for a number of representative

locations on the spillway channel and walls are shown on plates 133 and 134. It is significant to note that, for the most part, the movement trend that affected the spillway up to the 1960-1965 time frame did not start at the same time at all locations. A study of the plates indicates the movement in the lower portion of the spillway started while the channel was being excavated and, beginning at stations 21+00 and 24+00, movement was apparent at the end of excavation. At other locations, such as Station 7+00 and 11+00 and, for top of the wall at several other locations, the movement apparently did not start until 1939 or later. The maximum spillway movment was at Station 36+40. . These graphs indicate a maximum movement in the channel bottom of 0.08 foot per year and a movement of the right wall of 0.13 foot per year. The initial elevation check made in October, 1937 indicated about 0.3 foot of rebound for the approach slab. Since this, date the rate of movement has decreased. Beginning about 1943 or 1944, the rate of movement, while somewhat erratic, was essentially a straight line.

Plates 135, 136, and 137 show movement profiles along the channel centerline, at right and left toe, and at top of wall on the right and left sides. Plates 133 and 134 show profiles for 15 August 1939, 29 June 1943, and December 1949. This information is included to show the history of early movement. The profile for 1939 indicates that, in the wide part of the spillway channel, there was only slight vertical movement (compared to 1937) from Station 0+00 to Station 25+00, and for the 1943 profile there was only slight vertical movement from Station 0+00 to Station 16+00. This indicates that the movement along the centerline in this section of the spillway was quite late in starting. The 1949 profile indicates that a great share of the vertical movement has occurred since the date of the 1949 survey. The movement started along each wall and, in the wide section of the spillway, took a number of years before the movement reached the center of the spillway. In the lower portion of the spillway, the zone of influence from each side overlapped and caused maximum movement along the spillway centerline. The early profiles indicate that, for the lower portion of the spillway, movement was taking place at the time the concrete was poured. The center slab was poured to Station 50+00 during the 1936 working season, and

the initial movement observations, taken 7 October 1937, indicate a substantial amount of movement. The later area of maximum movement at Station 36+40 appeared very distinctly on this early survey. Plates 135-137 show the movement profiles of the top of the wall and toe of wall for the right and left sides of the spillway channel. Early movement profiles are included in order to study the start of movement with respect to the 1960 movement profiles. The original ground profile directly above the toe of the wall is shown for each graph. The profile of the channel slab and top of wall is included for reference to determine the amount of overburden removed at each wall. There does not appear to be a consistent relationship between the overburden removed and the amount of vertical movement.

9.3.3 Plate 132 shows that the height of slope was used for comparison with the observed movement. Indications were that there was a relationship between the height of slope on the right side of the spillway and spillway channel movement. Areas of maximum movement of the spillway channel centerline occurred slightly downstream of the points of maximum slope height. The areas of maximum movement were opposite areas of high cut slopes. Downstream of Station 36 the top of wall moved as a unit and appeared to bridge over areas of lesser potential movement.

9.3.4 It was evident, from a study of the sections at the upper end of the spillway channel (Stations 7+00 to 21+00), that movement was not directly related to the amount of overburden removed. A relationship was established between the height of side slope and the amount of movement for the spillway as a whole. The areas of maximum movement usually occurred opposite high side slopes. For the lower portion of the spillway channel, most of the high slopes that caused unusual movement were on the right side of the spillway channel. The shape of the movement pattern in the upper portion of the spillway channel seemed to offer an excellent example of plastic deformation due to the surcharge of high side slopes. There were two distinct upheaval areas, one along each side, on each section where the spillway section was wide. The movement pattern showed the progressive spread of the movement from the sides toward the center of the spillway channel. One explanation

was that in some areas the concrete channel could be carried upward faster than the foundation movement. This would leave the void beneath the channel floor that was found by test drilling for weak rock at Station 42+10. Starting at Station 19+00, and on downstream, the channel narrows and the upheaved areas met in the middle which usually resulted in maximum movement at the channel centerline typical of the movement pattern in the middle and lower portions of the spillway channel. The maximum movement in the lower spillway channel occured where the section was opposite a high side slope and where a minimum amount of overburden was removed above the spillway channel centerline. The shape of the movement profile for each section remained quite similar from the start of observations to 1962. Spoil has been placed in the concrete channel since 1962 and has added a considerable toe load which resists further movement.

9.3.5 A triangulation system to check for horizontal movement was started in 1939 at the time vertical spillway movement observations were started to provide a means of checking for the total combined transverse horizontal movement of both spillway walls at about 800-foot intervals. The graphical results of the triangulation movement study are shown on Plates 102, 122, and 123. A study of the horizontal movement graphs indicated horizontal movement was somewhat erratic but, over a long period of time, usually followed a straight line trend. The rate of movement showed a tendency to decrease at all measuring points except at Station 35 + 60 (I-J). Horizontal movement here continued at a steady rate. In 1949 an additional movement check was established to accurately determine the movement of the wall at more closely spaced intervals in the area of maximum movement along with the lower spillway. Graphs of this check indicated that major movement tended to decrease slightly at some locations up to 1955.

9.3.6 In the past, attempts were made to relate the areas of excessive movement to the fault pattern of the spillway area. In 1962 the original geology notebooks and drill logs were obtained from records retirement and an attempt was made to locate the faults in the areas of maximum movement - especially at Station 36+40. In the lower spillway area, data was available

from four core drill holes which were drilled in 1952. Notes of the 18 December 1935 geologic data stated that "the shale between 36+00 and 36+60 has begun to show signs of failure by cracks opening along joint planes adjacent to and along a fault plane which crosses the spillway centerline at 36+60." The note indicates movement was apparent at this location at the end of excavation and before pouring of the concrete channel slab. The elevation at the time of the inspection was probably a few feet above the present centerline as the shale was not excavated to final grade until just prior to pouring the channel slab. Data to relocate this fault could not be located in the original geology notes.

9.3.7 In the "History of Spillway Channel Construction," under Geology, Part III, mention is made of the undesirable effect of numerous bentonite beds near the base of the channel for the lower portion of the spillway. Wherever bentonite beds were found to be within two feet or less of finished grade, the foundation line was lowered and the bed removed. The 1952 drill records bear out the fact that bentonite seams are very numerous in the lower portion of the spillway.

In 1952, a hole was drilled at Station 42+10 (10 feet right of centerline) and a tiltmeter installed at this location to sample an area of minimum movement. While drilling through the concrete, a void of about 0.3 foot was found under the channel floor. From this evidence it was assumed that the channel slab had been lifted by areas of excessive movement on each side and was bridging across the area. The area containing the void was opposite minimum height side slopes on each side and is favorably located with respect to fault planes. Fault plane patterns indicated that the areas of excessive movement were adjacent to faults which dip toward a high cut slope. Movement from the high cut slope was interrupted by the fault plane and directed upward to produce unusual vertical movement. The area of excess vertical movement of the right wall also roughly coincided with fault planes that dipped toward the high side slopes. The vertical movement of the top of the wall along the left side of the channel was at a minimum about Station 34+50. At this station a slide occurred during construction.

After the retaining wall was constructed, fill for the roadway was placed behind the wall. The fill very likely caused a slight initial settlement of the wall. The movement varied from almost no movement next to the gate structure (and adjacent to the cellular wall construction opposite the training walls) to about one foot of movement at the downstream end of the roadway retaining wall, which is near station 9+00 (spillway stationing). The movement of the end of the wall was about the same as the vertical movement of the channel wall on the east or right side and less than the channel wall movement on the left side or west side. As of 1962, the movement changed from no appreciable movement at the gate structure to the maximum movement of about one foot for the end of each wall. Movement was decreasing.

9.3.8 The differential movement at the construction joint and the downstream edge of the gate pier foundation was a total of 0.2 foot by November 1940. The training wall slab is keyed to the gate pier foundation so the difference in movement probably represents strain of the concrete. Field checks in 1944 and 1945 indicated the key had failed at piers three and four where the differential movement was greatest. Since this time, there has been no further apparent failure of the key. Movement points on the training wall slab show a fairly constant rate of increase, while the points on the gate pier foundation show a much slower and erratic movement since 1940. As of 1962, the total average differential movement at the joint was only about one-tenth foot.

9.3.9 Plate 140 shows a study of spillway movement in a longitudinal direction from the upstream edge of the approach slab through the gate structure to the end of the training wall slab. The movement shown is for November 1960 and indicates the difference in movement that occurred between the more heavily loaded and deep seated gate pier foundation and the lightly loaded slabs upstream and downstream of the gate structure. This profile also indicates the upstream tilt of the gate structure.

9.3.10 The area immediately downstream of the lined channel is of special interest in connection with movement observations. The depth of scour and the erosion of the high shale slopes have had a direct bearing on slope stability and the structural stability of the end of the lined channel. Since initial spillway discharge in 1946, an enlarged natural stilling basin section has been eroded. The maximum depth of scour at the end of the spillway is about 26 feet below the end of the lined channel. This deep scour area slopes upward and intersects the original excavation line 450 feet downstream from the end of the lined channel. The indications are that this natural stilling basin has become fairly stabilized and further enlargement will proceed very slowly because under normal anticipated operations, future discharges through the spillway will be very infrequent and of limited duration. The lined concrete channel is protected from undermining by a concrete cutoff wall that extends 70 feet into bedrock. No movement has occured for the cutoff wall.

9.3.11 Several crack surveys of the spillway channel floor were made after movement observations started. Plate 86 shows the 1939 crack survey data. From a study of the crack pattern and from field inspections, most of the shorter random cracks which were mapped were contraction cracks. The cracks which seemed to make a pattern and extend from one block to the next were usually found in the areas of maximum movement. The most significant cracking occured on the approach slab about 20 feet upstream of the gate structure and the diagonal cracking pattern beneath Stations 36+00 and 37+00. Other significant cracks were found adjacent to areas of unusual movement. In September of 1959, a large crack was noted on the approach channel side walls on both sides of the spillway. A study of the spillway plans indicated that cracks in this area are just opposite the upstream end of the approach channel floor slab. The rupture of concrete for this joint indicates that the expansive force of the shale must be considerable. The crack could have been caused by the Yellowstone earthquake (August 1959) which was felt at Fort Peck.

9.3.12 Detailed information and data conceiving historical movement trends at the spillway are contained in the study, "Report on Spillway Movement Observations, Fort Peck Dam," Omaha District, Corps of Engineers, September 1962. The report contains 25 years of data (1937-1962) and indicates that there was substantial movement in the past. Some of the movement could be attributed to rebound due to removal of overburden. However, movements that were of special concern were differential movements in localized areas. This type of movement was steady and did not appear to slow down with time. As of 1962, indications were that such movement would continue.

9.3.13 A subsurface exploration program was initiated in April 1963 to obtain more details of geologic conditions. A total of thirty-three holes were drilled from about Station 33+00 downstream to about Station 43+00. Hole locations are shown on plates 92 and 93. Graphic logs are shown on plates 94, 95, 96, and 97. Tiltmeters were placed in some of the exploration holes. Tiltmeter locations are shown on plate 92. Two additional tiltmeters were installed at a depth of over 200 feet in 1965. A crack survey (plate 93) was conducted in 1963 and surface movement pointers were set (plate 92). The results of geologic mapping of the cut slope on the right side of the spillway are shown on plate 93.

9.3.14 In addition to the investigations that have previously been discussed, the 1963 investigations also included: Spillway movement vs. slope height evaluations, laboratory analysis of subsurface samples, slab replacement study and spillway under-drain studies.

9.3.15 The results and recommendations of the complete study has been published as Omaha District's Design Memorandum No. MFP-109, September 1966, "Spillway Rehabilitation."

Plate 129 of the present foundation report shows the results of the slope movement studies. The points were set three feet into surface weathered material. Tiltmeter observations are given on plates 140 through 143.

Plate 132 shows the correlation between spillway movement and slope height. The data is for representative sections along the length of the spillway at one to two hundred foot intervals from Station 5+00 to Station 41+00.

9.3.16 As of 1969, observations indicated that movement in the spillway area was continuing at the same rate as in the past and probably would remain a continuing problem.

Tiltmeter observations had clearly established that a constant rate of horizontal movement toward the spillway channel was taking place across certain bentonites. This horizontal movement was interrupted at certain locations by faults which changed the horizontal movement to vertical movement along the fault trace. These areas of excessive differential vertical movement in each case seemed to center on rather low angle dip faults which cut across the spillway channel at an angle. The area of maximum differential movement was at approximately Station 36+00. Another area of severe differential movement centered on a fault which crossed the spillway at approximately Station 40+00. Excavation of slope adjacent to the spillway was undertaken in order to stop movement in these areas of the structure. Excavation was in two stages.

9.3.16.1 Stage I Slope Excavation: The area opposite Station 36+00 was the area selected for the first experimental slope excavation contract. The grading plan as finally approved consisted of cutting a wide berm 40 feet above the spillway berm road with 1V or 5H backslope. No grading was to be performed on the immediate 1V on 2H slope above the berm road under this contract as it was desired not to grade immediately adjacent to the spillway channel. The width of the berm was designed so it would be possible to construct final slopes above the berm road of 1V on 5H with the drainage berm 40 feet above the spillway berm road. The Stage I experimental slope excavation contract was started during January 1969 and was completed by mid-August 1969. Following an inspection in 1969, it was decided to extend slope excavation further downstream to Station 40+00 (Stage II excavation).

9.3.16.2 Stage II Slope Excavation: For Stage II excavation, it was decided to grade the slope above the berm road to 1V on 3.5H, and this grading was extended back to cover the Stage I area.

9.3.16.3 Results of Slope Excavation: Tiltmeter observation along the berm at Stations 35+00 to 36+00 indicated an average rate of movement of .087 foot per year prior to Stage I excavation. After excavation there was a gradual slowing down, and by 1973, the rate averaged about .03 foot per year or about one-third the rate prior to slope excavation. The horizontal movement at Station 40+00 was divided between two bentonite layers. The lower bentonite movement was measured by a tiltmeter on the berm, and the rate of movement for five years prior to Stage II slope excavation was .029 foot per year. The slope excavation was completed between July and November 1970 and some motion continued to June 1971. Since this date, there has been no measurable movement across the lower bentonite. Prior to Stage II excavation, the rate of movement across the upper bentonite as measured by the above tiltmeter was .025 foot per year and since completion of excavation seems to have stopped completely. The movement across the upper bentonite ("M") adjacent to the berm road is measured by a dial gage. Prior to excavation, the average yearly rate was .027 foot per year and since about April 1971, the rate has averaged about .006 foot per year or about 22% of what it was prior to slope excavation. Since 1973, the observation from the tiltmeter installations and the dial gage installation indicate a definite decrease in rate of movement, and regular movement surveys also indicate the same trend.

9.4 POWERHOUSE FOUNDATION MOVEMENT.

9.4.1 Movement Points: Movement points were established on the foundation structure of Powerhouse No. 1 at the completion of the substructure contract. The location of the movement points are shown on the plan on plate 172. The movement points were observed at frequent intervals during the period of construction and have been observed at infrequent intervals since the completion of the powerhouse. The movement points for Powerhouse No. 2 were established on the substructure at about the time the superstructure contract was

started. Additional temporary movement points were established at the generator floor elevation during the time of machinery installation for special tilt studies. The location of movement points for Powerhouse No. 2 are shown on the plan on Plate 178. The permanent points on the substructure foundation and on the draft tube deck are available for future observations.

9.4.2 Movement Records: The graphical time movement records of movement points for Powerhouse No. 1 are shown on Plates 172, 173, 174, and 175. The plan at the left shows the location of all movement points and the reference bench marks. The individual graphs show the movement record for movement points where continuous observations have been made. Most of the movement points in the penstock area were destroyed at the start of surge tank construction. The graphs indicate there was some variation of movement, but, in general, fairly definite trends existed. The initial settlement during the construction of the powerhouse superstructure was expected, except for some of the draft tube points. From 1944 through 1950, the movement was quite erratic, but a general movement pattern or trend could be noted. Minor variations in the trend could possibly be explained by localized loading, foundation adjustment, slight movement of the reference bench mark, as well as the inherent errors of survey. An average line can be drawn through the data from 1944 to 1950, and the line in some cases can be continued to the present. However, most graphs indicate a decreasing trend since the end of construction. The graphs indicate considerable movement between January 1950 and June 1951. There was rebound of the draft tube movement points and an opposite settlement of most movement points on the foundation upstream of the draft tube points. It appears that a foundation adjustment took place during this period. This movement is further discussed in paragraphs 9.4.3 and 9.4.15. The Initial observation for movement studies (August 1941) was at the completion of the substructure and prior to the start of the contract for the superstructure and installation of No. 1 turbine and generator. The profile for September 1944 (Plate 174) shows the movement for the powerhouse with Unit No. 1 operating without surge tanks. The profile for November 1949 (Plate 174) indicates the effect of the surge tanks and water load. At this date, Unit No. 2 and the surge tanks had been in operation for a little over

one year. The October 1962 (Plate 174) profile shows the powerhouse with all three units in operation. The installation of Unit No. 3 during 1951 had little effect on foundation movement. The transverse tilt sections shown below the plan on Plates 174 and 177 indicates there has been slight change in tilt across the units since 1944, but the tilt sections shown below the typical powerhouse section (Plates 175 and 177) indicate an increase in tilt over the years. The time tilt graphs to the right were prepared to study the tilt for all observations. The two points used to compute the tilt are indicated on each graph. The tilt was reduced to tilt in inches per foot so the tilt of the units of Powerhouse No. 1 could be compared directly with the tilt of the units of Powerhouse No. 2. It should be noted that, after 1953, the Fort Peck Reservoir was drawn down about 80 feet below maximum level and as of 1963 was about 50 feet below maximum level. At the lower lake level, the surge tank water load was considerably less and could account for some of the apparent decrease in tilt at this time. Unit 1 has a measured tilt averaging 0.0090 inches per foot since 1963; however, this figure amounts to only 0.0065 inches per foot since the machinery installation period. Tilt of 0.0065 inches per foot equates to 1 part in 1850, which is well within the equipment's tolerance. Similiar data presented for the second power plant is shown on Plates (176, 178, and 179). Unit 4 has tilted approximately 0.005 inches per foot since 1961 when the machinery was installed; this equates to 1 part in 2000 and, again, is well within the equipment's tolerance for tilt.

9.4.3 Early Unusual Movement: After completion of the outlet works and prior to construction of the Powerhouse No. 1 foundation, it was found that the outlet floor slab had heaved a maximum of about one-half foot. The floor slab adjacent to the wall is cantilevered to the wall and remained relatively stable. The floor slab movement in the vicinity of tunnel outlet portal No. 2 caused a rupture of the concrete in the cantilever floor slab. This evidence of unusual movement is at the approximate location of the downstream right corner of powerhouse No. 1 which has also shown more than normal upheaval. It is possible that the movement of this corner of the powerhouse could be partly caused by high foundation pressure transmitted to this particular area by fault blocks which concentrate movement at a particular location.

9.4.4 Powerhouse No. 2 Movement Observations: The time movement graphical records for Powerhouse No. 2 are shown on Plates 176, 178, and 179. Included on Plates 178 and 179 are the movement contours for the October 1962 observation which indicates the general pattern of movement. The Superstructure and surge tank construction contracts were started during early 1959. The initial construction of the powerhouse superstructure and surge tanks caused rapid initial movement. The surge tanks were initially filled during September of 1960. Movement observations were made at frequent intervals to record the effect of the water load. There were no observations between 26 September 1960 and April 1961. During this period, the surge tanks were filled with water while the turbines and generators were installed. Thus, most of this time period represents full surge tank water load on the foundation. During May of 1961, the surge tanks were unwatered just prior to starting up the units. The period since May 1960 represents normal operation of Powerhouse No. 2. The rate of movement of the upstream points on the surge tank foundation has decreased since this date, and the movement of the downstream points on the draft tube deck indicate a reversal of movement. The rebound of the downstream points indicates a similarity to the movement experienced in Powerhouse No. 1, except that for Powerhouse No. 1 the tendency was indicated from the start of construction.

9.4.5 Summary of Tilt Observations: The movement profiles on Plate 176 summarize the tilt experienced during construction and initial operation of Powerhouse No. 2. An initial zero date of 2 May 1959 was used for the tilt studies. All tilt is relative to this date which was after the start of superstructure construction. The profiles show the movement at significant dates during construction. The profile for 27 May 1960 shows the tilt about one year after the intial zero date of 2 May 1959 and includes the period in which the scroll cases were embedded. The movement profile for 22 September 1960 shows the effect of the surge tank water load. The surge tanks had been full for 24 hours at the time of this observation. It should be noted that the water load for this test exceeded the normal water load during operation of the units. The profile for 8 May 1961 shows the tilt at the time the units were on test run. This profile is used as a standard to measure

subsequent tilt after the units were placed in operation. The graphs indicate tilt was somewhat erratic, but over the entire period of construction, and initial operation, there was in general a progressive increase in tilt. The construction of Powerhouse No. 1 (after substructure) extended over a ten-year period while Powerhouse No. 2 (after substructure) was completed in less than three years.

9.4.6 Special Movement Observations: Plates 178, 179, and 183 show the results of special loading and unloading tests of the surge tanks for Powerhouse No. 2. Temporary movement points were installed in the generator bay at elevation 2063.5 msl to observe the results of the tests. The test during September of 1960 was the intial filling of the surge tanks. The average tilt across unit 4 and 5 due to the water load as measured by the permanent points was .0020 inch per foot. For the temporary points, the average tilt was .0017 inch per foot. The movement profiles for 26 September 1960 show the tilt after the surge tanks had been dewatered three days. The profiles indicate the foundation tilt decreased with the unloading but did not quite return to the original position. The agreement between the permanent and temporary points was very good.

Between the test of September 1960 and the test of May 1961, the surge tanks were filled with water and kept full while the turbines and generators were installed. At the completion of the machinery installation, an unloading test was made to determine the amount of tilt that unloading of the surge tanks would cause. For this test, eight temporary movement points were installed on the generator bay floor at 2063.5 elevation. The movement profile for 1 May 1961 shows the tilt after the surge tanks had been dewatered. The agreement between the temporary and permanent points was extrememly good. The average downstream tilt for unloading as measured by both the temporary and permanent movement points was .0006 inch per foot. The surge tanks were again filled with water and the units were put on mechanical test run. The movement profiles recovery of both temporary and permanent points for Unit No. 4 was good. The unwatering test can be considered a standard for the probable amount of downstream tilt that can be expected when the surge tanks are dewatered in the future.

9.4.7 Summary of Powerhouse Movement: As of 1963, powerhouse movement observations covered a period of about 21 years for Powerhouse No. 1 and a period of about 4 years for Powerhouse No. 2. In some cases, the experience with Powerhouse No. 1 helped to confirm indicated movement trends of Powerhouse No. 2. A study of the time movement records indicated a maximum settlement at the end of construction of about .06 foot for Powerhouse No. 2. Using November 1949 as a comparable date for Powerhouse No. 1, the maximum settlement at the upstream edge of the surge tanks foundation was about .04 foot. The rate of movement for Powerhouse No. 2 was much greater than for Powerhouse No. 1. The maximum rate as measured by an average curve was about .026 foot per year. The maximum rate for Powerhouse No. 1 appears to be less than .010 foot per year.

9.4.8 The average movement graph for Powerhouse No. 1 indicates erratic settlement to 1949 with a rate of settlement of about .002 foot per year from 1941 through 1949. The total average settlement of Powerhouse No. 1 is about .015 foot of settlement and seems to have remained at approximately this value since 1949. The differential movement for the entire foundation structure now exceeds .080 foot from downstream right corner to upstream left corner.

9.4.9 The average movement graph for Powerhouse No. 2 indicates .040 foot of movement to the end of construction (May 1961) and no apparent trend since this date. The rate of settlement was about .018 foot per year. The differential movement of Powerhouse No. 2 was about .050 foot from the downstream right corner to the upstream left corner. All movement points on the downstream edge of the structure indicate that upward movement occured and seems to agree with movement experience for Powerhouse No. 1. The upward trend of the downstream points for Powerhouse No. 1 was evident from early construction, while for Powerhouse No. 2 the rapid loading of the foundation caused an over-all settlement, and the upward movement trend of the downstream points only became evident since the end of construction.

9.4.10 The tilt of Powerhouse No. 1 caused no apparent difficulties in the installation of turbines and generators while Powerhouse No. 2 required

regrinding of seats before the turbines and generators could be installed. The tilting was erratic, but the over-all graphs indicate definite average trends. The rate of tilting for Powerhouse No. 2 averaged four to five times the rate of tilt of Powerhouse No. 1. The total tilt (expressed as inches per foot of length) of Powerhouse No. 1 at the end of construction exceeded the total tilt of Powerhouse No. 2.

9.4.11 Comparison of Foundation Conditions and Other Related Factors: The foundation design for Powerhouse No. 2 was patterned closely after Powerhouse No. 1 on the assumption that No. 2 would perform similarly to the behavior recorded by some 12 years of observations then available for No. 1. Plate 186 is a plan of the outlet structure and the adjacent abutment area with the location of the powerhouses superimposed. The locations of all exploratory borings and the trace of all faults mapped during the excavation of the outlet structure and the tunnels are shown on the plan. The faults in the outlet area are shown on the 2023.82 plane on Plate 181, which is the base of the outlet floor slab, and the faults in the tunnel area (mapped during tunneling) are all shown on the 2040 elevation plane. The dashed lines (Plate 181) in the outlet area are the March 1945 movement contours for the rebound of the outlet floor slab (slab installed 1935). This is the earliest survey data available. The movement pattern for the outlet structure indicates that rebound may be closely related to the fault pattern. Most of the faults in the areas of maximum movement dip toward the high abutment slope. The movement on the abutment side of the fault appears to be maximum and the movement decreases rapidly on the opposite side of the fault (toward the valley). The arching action of the concrete floor slab masks the shale rebound and probably causes the apparent movement on the valley side of the faults. Some of the faults branch with legs of faults curving toward the abutment. It appears that maximum rebound occurs in these areas. Most of the upstream foundation area for Powerhouse No. 1 appears to be free of faults, and the outlet floor slab is narrow at this location so the original rebound could have been reduced by the confining effect of the two sidewalls and backfill.

9.4.12 Powerhouse No. 2 is located in an area of considerable faulting, and the movement contours indicate it is an area of maximum rebound. The area was unloaded for about 23 years before the start of construction for Powerhouse No. 2. The area of minimum settlement for Powerhouse No. 2 compares to the area of maximum rebound for Powerhouse No. 1 (downstream right corner). The rebound contours for the outlet floor slab change from maximum movement to minimum movement in a very short distance adjacent to the right downstream corner of Powerhouse No. 2. This could suggest that stresses which caused the original rebound were transmitted to this side of the powerhouse foundation and tended to reduce settlement.

9.4.13 Plates 184, 185, and 186 are included to compare the foundation conditions of Powerhouse No. 1 and No. 2 with respect to the more general features which could effect foundation movement. The locations of the sections are shown on the plan of Plate 184. The sections on Plates 185 and 186 show the main geologic features such as marker beds, faulting, present slide plane in slide area, original ground line, 1950 ground line, (which is essentially the same as at present), and the trace of the shale line as it dips beneath the valley flood plain. The sections show the main bentonite marker beds in the area as found by exploratory borings and the mapping program carried out during outlet and tunnel excavation. The displacement and change of dip of the marker beds and the available fault data indicate faulting in the area is quite extensive. The original ground line and the present ground line (and structure excavation line) allows a comparison of the amount of overburden removed at each powerhouse site. The excavation at each foundation area was comparable. However, for each powerhouse foundation the excavation along the right edge of the foundation exceeded the excavation along the left edge.

9.4.14 Comparison With Other Observed Foundation Pressure: The Bearpaw shale is a compact marine shale which was heavily preconsolidated, and for construction purposes, is regarded as a weak rock. However, when used for a foundation for a heavy structure without piling, it could be expected to behave, to a certain extent, in the same manner as less compact clay shale.

A pressure cell installation under Unit No. 2 of the Garrison Powerhouse, which is founded on the clay shale of the Fort Union Formation, indicated that pressure increased toward the edge of the powerhouse foundation. Reference should be made to "Interim Report on Powerhouse and Stilling Basin Pressure Cell Observations Garrison Dam," dated December 1961. The study indicated that, initially, the foundation stress is fairly evenly distributed but, as time goes on, the edge pressures tend to increase, and pressures in the middle of the foundation slab actually decrease. This could possibly explain the unusual movement experienced in Powerhouse No. 1 between June 1950 and January 1951. The explanation would require strain or cracking of the foundation in order to relieve high edge pressure.

9.4.15 Other Miscellaneous Factors: The unusual movement of Powerhouse No. 1 between June 1950 and January 1951 has been mentioned previously in this report. It has been suggested that earthquake movement could have been a contributing factor to the adjustment. During 1959 several cracks were discovered in the concrete channel upstream of the Fort Peck Spillway, and it was considered that these cracks could have been a results of the Yellowstone earthquake of August 1959 which was felt at Fort Peck. A letter of inquiry to the Montana School of Mines revealed there were two earthquakes in the western mountain area between June 1950 and January 1951 with a Mercalli Intensity of 6. These earthquakes are listed as follows:

27 June 1950 - Yellowstone Park area with epicenter at 44-3/4°N, 110½°W

19 Aug. 1950 - Western Montana with epicenter at 47-1/4°N, 113½°W

Earthquake data is cited as a possible cause or source of the unusual movement experienced during this period. No other unusual movement was noted for structures at Fort Peck during this time period.

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